DESIGN COMPARISON OF PRESTRESSED VS. POST-TENSIONED PRECAST CONCRETE BRIDGE BEAMS

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POST-TENSION PRECAST CONCRETE BRIDGE BEAMS

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ABSTRACT

DESIGN COMPARISON OF PRESTRESSED VS. POST-TENSIONED PRECAST CONCRETE BRIDGE BEAMS

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The principal objective of this thesis is to investigate the status and techniques of post-tensioning and splicing precast concrete I-beams in bridge applications. Representative projects are presented to demonstrate the application and success of specific methods used. To demonstrate the benefits of using post-tensioning and splicing to extend spans, multiple analyses of simple span post-tensioned I-beams are performed varying such characteristics as beam spacing, beam sections, beam depth and concrete strength. Design Tables are developed to compare the maximum span length of a prestressed I-beam versus a one-segment or a spliced three-segment post-tensioned I-beam. The lateral stability of the beam during fabrication, transportation and erection is also examined and discussed. The tables are intended to aid designers and owners in preliminary design studies to determine if post-tensioning can be beneficial to their project at hand. AASHTO Standard Specifications are used as basic guidelines and specifications. In many cases, the results indicate that post-tensioning extends the maximum span length of a typical 72-inch deep precast I-beam by more than 40 feet over conventional prestressed I-beams.

TABLE OF CONTENTS

APPENDICIES

LIST OF FIGURES

LIST OF TABLES

CHAPTER I

INTRODUCTION

Beginning circa 1950, precast concrete construction comprised only 2 percent of all bridges in the United States. Today, precast concrete is used in more than 50 percent of the nation's bridges (Rabbatt, et al, 1999). Economical fabrication costs, rapid production, widespread availability, low maintenance, and a long term life cycle are the primary reasons for such an increase in its use. Prestressing is the common choice for reinforcing precast beams; however, prestressing requires an entire span length to be transported in one piece and without intermediate splices. As a result, the maximum span length of prestressed beams has been limited to a maximum of 160 feet. Although long spans have been constructed using such methods as segmental cantilever construction, cable stays, and segmental arches, these methods require complex analysis, special construction techniques, and utilize custom precast sections, all of which are expensive and do not lend themselves to mass production. Recently, a new method of precast construction is emerging that extends precast concrete bridge spans into the 160 foot to 300 foot range that was previously dominated by steel plate girders. The construction method overcomes the transportation limitations by utilizing post-tensioning in conjunction with prestress. Combining post-tensioning and prestressing allows multiple sections to be spliced together resulting in longer precast spans.

The most efficient beam cross-section for prestressed concrete is an I-beam configuration. A precast I-beam has a wide top flange, a thinner web, and a wide bottom flange also referred to as the bulb. This bulb enables as many as 70 prestressing strands to be placed in the bottom of the beam to serve as reinforcement. The same precast I-beam cross-sections can be used when post-tensioning and splicing construction methods are applied. Splicing can be used in single span or multiple span applications.

In a single span post-tensioned bridge, either a single segment or three individual segments are cast with a constant cross-section. Post-tensioning ducts are aligned in a parabolic alignment from end to end of the finished beam. Single-segment beams are transported to the site, set on their final supports, and post-tensioned longitudinally. Post-tensioned beams longer than 160 feet typically require the beam be divided into three sections for transportation. The use of three segments allows the field splices to be placed away from the midspan, where the moment stresses are greatest. The three-segment construction method requires temporary supports. The beams are spliced together in the field atop intermediate temporary supports and are post-tensioned longitudinally. Once the splice is made and at least one tendon is pulled, the temporary supports are removed. See Figure 1.1 for an illustration.

Multiple-span bridges typically utilize precast segments that are continuous over the piers in the negative moment region. This segment is commonly referred to as the "pier segment". The segment that connects the pier segments is primarily subjected to positive moment stresses. This segment is often referred to as the "drop-in" section because it is the last beam segment to be erected and "dropped in" between the two pier segments. Constructing multi-span bridges in this manner places the splices at the approximate points of contraflexure where the stresses are minimal. The end segments typically span from the abutment to the first point of contraflexure. As a result, a typical two-span bridge uses three precast segments along its length, and a three-span bridge uses five segments along its length. An added advantage to this type of construction is that the negative moment section over the pier can be varied or "haunched" to handle the high

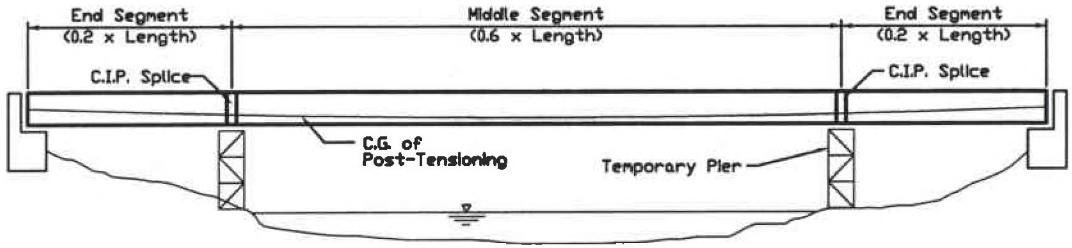


FIGURE 1.1
Single-Span, Three Segment,
Post-Tensioned Bridge Layout

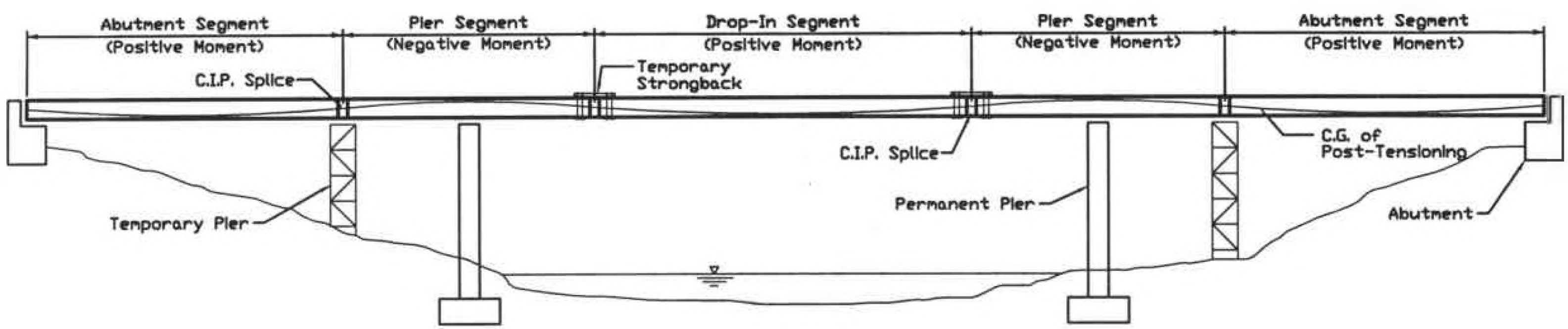


FIGURE 1.2
Three-Span, Five-Segment,
Post-Tensioned Bridge Layout

stresses that result over a pier. Multi-span bridges with continuous beams over the piers can potentially eliminate several if not all of the temporary supports. This can be accomplished by constructing temporary moment connections to rigidly attach the beams to the pier. Then temporary connections such as "strong-backs" or "Cazaly hangers" are employed to hold the drop-in segment in place until the permanent splices can be made. Figure 1.2 is an illustration of a multi-span bridge that uses a combination of temporary supports and temporary connections.

CHAPTER II

REPRESENTATIVE PROJECTS

The technology of post-tensioning and splicing precast I-beams has been in use almost as long as prestressing; however, it is used in less than 1 percent of the bridges in the United States. The use of post-tensioned and spliced concrete beams has been confined to certain regions of the country, such as Florida and Utah. Most states have constructed only a few post-tensioned or spliced beam bridges with many having none at all. Owners and designers have been reluctant to use post-tensioning and splicing because it requires a complex analysis, a more skilled contractor, and fabrication and construction techniques that are more complex than current standard procedures. Consequently, it is difficult for owners and designers to assess the economics of such construction methods. However, when given the choice, many contractors have elected to use post-tensioned and spliced beam bridges because of their advantages. Innovative building procedures such as "design-build", "value engineering", and "multiple alternative bidding" have allowed these designs to happen. Contractors who acquire the materials, pay the laborers, furnish the cranes, and build the bridges are often better able to assess the most efficient type of bridge at a particular site. Often, the contractor is telling designers and owners that post-tensioning and spliced precast beams are the least cost alternative to build. Below are a few examples of such cases:

Design-Build- With the 2002 Winter Olympics taking place in Salt Lake City, the Utah Department of Transportation (UDOT) recognized the need for replacement or widening of more than 130 bridges to accommodate the added traffic. This also had to be done fast with the strict deadlines. UDOT utilized design-build to select a designer-contractor team that could meet this deadline at the least cost. One of the challenges for the designated team was to design 17 single-span bridges 210' to 220' in length that would span over single point urban interchanges (SPUI's). The team selected spliced precast I-beams over steel plate girders to reduce both cost and fabrication time. The precast I-beams used were 94" deep and spaced over 10 feet apart. (Schutt, 1999).

Multiple Alternative Bidding- The Ohio Turnpike Commission needed to replace the twin steel truss bridges that span over the Cuyahoga River Valley. The cost of replacing these bridges, each over 2,600 feet in length and 175 feet over the water at their highest point, were going to be substantial. To control cost, the Ohio Turnpike Commission requested both steel and concrete alternatives to be designed and bid. The concrete alternative utilized spliced beam technology and post-tensioning while the steel alternative utilized plate girders. The winning bid was the concrete alternative with a bridge construction cost of $38.5 million, $1.4 million cheaper than the steel alternative. Of the six contractors who bid the project, four of them submitted bids that were less than the steel alternative. Another important aspect of this type of bidding is that it forces the steel and concrete fabricators to offer more competitive prices to the contractors compiling the bids since they must also compete with another material. If either steel or concrete had been the only material to bid, the material costs would likely have been higher. (OTEC, 2001).

Value Engineering- As part of a project to widen Cincinnati-Dayton Road from two lanes to five lanes, the curved bridge that carries this road over I-75 was to be completely replaced. The original consultant chose to design a four span, curved, steel, plate girder bridge. Conventional Prestress I-beams could not be used because the middle span length of 130 feet did not allow sufficient vertical clearance over I-75 without major adjustment to the vertical profile of one or both roads. The contractor that won the project chose to utilize a value engineering approach so that a precast concrete solution could be used. The contractor's consultant redesigned the bridge using post-tensioned precast beams. Using the same span arrangement and number of beam lines as the steel girder, the precast concrete I-beams were designed without intermediate splices since the 130 foot beams were easily transported. An angle point at each pier accommodated the curved alignment. Once set into place, the beams were post-tensioned longitudinally for superimposed dead loads and live loads. Though a special I-beam section was developed employing a 4′-0″ wide by 5″ thick top flange, the 4′-0″ deep I-beam was only 1″ deeper than that of the steel plate girder section at the pier for which it replaced. (ODOT/LJB Inc., 1999 and ODOT / Janssen & Spanns Engineering, Inc., 2001).

A questionnaire was sent to every state department of transportation to find out about projects in each state that have been designed utilizing post-tensioning and splicing methods. Of the 50 states, 22 states responded to the questionnaire. Of those 22 respondents, 11 states had at least one post-tensioned and/or spliced bridge with some states having multiple bridges constructed in this manner. Sixty-four questions were asked including the geometry, section dimensions, post-tensioning information, splicing techniques and construction methods. Table 2.1 summarizes the general characteristics for each bridge. In addition to these bridges, there are many other structures both in the states that responded and in states that did not respond. No attempt was made to include these bridges in the results. Nevertheless, the responses received represent a cross-section of applications that demonstrate real uses and benefits of post-tensioning and splicing precast beams. (Appendix A is a sample of the questionnaire sent to each state).

TABLE 2.1
Summary of State DOT Survey of Bridges Designed Utilizing Post-Tensioinig and Splicing

State*	Roadway	over	Contract** (DBB, DB or VE)	Spans	Span Length Range	Tallest Pier	Beam Spacing	Beam Depth
Georgia****	N/A	Intercostal Waterway	DBB	20	180'	150'	7.50'	90"
Illinois	40A	FAI 72	DBB	2	106'		6.00'	48"
Louisiana	Colombia	Ouachota River	DBB[S]	N/A	250'	N/A		72"-120"
Louisiana	Jonesville	Black River	DBB[S]	N/A	250'	N/A		72"-120"
Louisiana	New Orleans	Rigdlets Pass	DBB	N/A	250'	N/A		72"-120"
Minnesota	T.H. 101 WB & EB	C.S.A.H. 16	DBB[C]	1	170'	-	5.27'	81"
Nevada*****	N/A	N/A	DBB	2	110'-104'	23'	9.20'	39"
Nevada*****	N/A	N/A	DBB	2	130'-120'	20'	9.50'	51"
North Carolina	US 64	Croatan Sound	DBB	3+	138'-230'	66'	9.91'	78"-132"
Oregon	N/A	Stream	DBB	2	160'	25'	7.83'	84"
Texas ***	US 183	McNeil Road	DBB	3	90'-140'	18'	7.35'	54"
Utah	I-15	4500 South	DB	1	211'	-	10.83'	95"
Virginia	Route 123	Occoquan River	DBB	7	144'-240'	49'	8.79'	N/A
Washington	I-5/SR 20	Cascade Mountains	DBB	1	197'	-	6.89'	95"

N/A Information Not Available

* *Connecticut, Delaware, Hawaii, Maryland, Massachusetts, Montana, New Hampshire, Oklahoma, Pennsylvania and Vermont DOT's also responded but indicated that to their knowledge, bridge projects of this description have been constructed in their state in the last 10 years.*

** *DBB, DB or VE indicates the method of contract. DBB indicates Deign/Bid/Build, DB indicates Design/Build and VE indicates Value Engineering. A superscript letter behind the conctract method indicates that the bridge was bid against a steel alternative. The supersript designates whether Steel or Concrete was the lowest bid.*

*** *Indicates splice only (No post-tensioining)*

**** *Indicates post-tensioning only (No splice other than at pier)*

***** *Indicates U-Beam Section Used.*

\+ *The US64 bridge over the Croatan Sound has three mainspans that are post-tensioned and spliced, there are actually 268 spans in total*

CHAPTER III

LATERAL STABILITY OF LONG PRECAST BEAMS

Design of long precast beams, whether prestressed or post-tensioned, cannot be properly addressed without considering the stability of the beam. Instability of the beam during fabrication, transportation, and erection can result in cracking or failure of the beam prior to the application of any external loads on the beam. Many variables contribute to the stability of a beam. It is beyond the scope of this thesis to go into this subject in great detail. Nonetheless, certain characteristics of the beam are essential to the stability of the beam and are discussed herein. The lateral moment of inertia of the beam is significant in resisting the torsion and weak axis bending situations that occur in long precast bridge beams. Other factors such as the amount of prestressing, the width of top and bottom flanges, strength of the concrete and length of the beam are crucial to the stability of the beam. Typical values of tilt are assumed when evaluating the stability of beams for this research. A factor of safety of at least 1.5 is typically desirable; although, beams with lower factors of safety can be safely handled by adjusting the location of support, increasing the stiffness of the supports, or utilizing stiffening trusses to name a few. A thorough discussion on the lateral stability of precast beams is described in PCI Journal articles. (Mast, 1989 and 1993).

The contractor and the fabricator are ultimately responsible for handling, transportation, and erection of the precast beams. If a precast beam has a low factor of safety, the burden of overcoming the handling issues is typically placed on the contractor and fabricator. A contractor bidding on a bridge project must build the cost of these corrective measures into his price, thus driving up the total project cost. For this reason, it is better for the designer to carefully consider these handling issues in the initial design of long precast beams. Sometimes, it is as simple as increasing the width of the top flange. This is often suitable for prestressed I-beams between 120 feet to 160 feet. If this is not feasible or does not fully correct the stability, post-tensioning can be utilized. Post-tensioning reduces the initial stresses in the beam until it has been erected and allows the concrete time to gain additional strength. For beams beyond 160 feet in length, splicing in conjunction with post-tensioning may be the only other alternative.

CHAPTER IV

DESIGN COMPARISON OF PRESTRESSING VS. POST-TENSIONING

A. Objectives

Post-tensioning and splicing techniques can be used to design a bridge that spans further, uses less girders, and/or has a shallower depth than by using prestressing alone. Unfortunately, owners and designers are reluctant to use post-tensioning and splicing on a regular basis because of lack of design standards and the complex analysis that is required. The cost of even exploring post-tensioning and/or splicing requires time, effort, and coordination that typical projects usually cannot afford. Though investigating the techniques of post-tensioning and the status of it's use are objectives of this thesis, the principle objective is to provide designers and owners with quantifiable limits of what post-tensioning and splicing can and cannot do. For comparison, the maximum spans of prestressed I-beams are compared to that of post-tensioned I-beams with all other variables held constant. This study also demonstrates the affect that changing various beam characteristics has on the maximum span length of a given section. These variables are as follows:

- Section Properties – Various standard 72" deep I-beam sections by region are analyzed.
- Concrete Strength – Three different common concrete strengths are analyzed.
- Beam Depth – 60", 72" and 84" beams of a given regional beam series are analyzed.

The information and tables developed in this thesis will allow designers and owners to more easily determine if post-tensioning and splicing is applicable to their particular project at hand. It will also aid the designer an owner in altering, if necessary, various beam characteristics to customize the capacity of the beam as needed.

B. Parametric Studies

For every trial, all dimensions and loadings are held constant. Table 4.1 below shows the loadings and bridge configurations used in the analyses. Only beam sections, beam spacings and concrete strengths are varied for comparison.

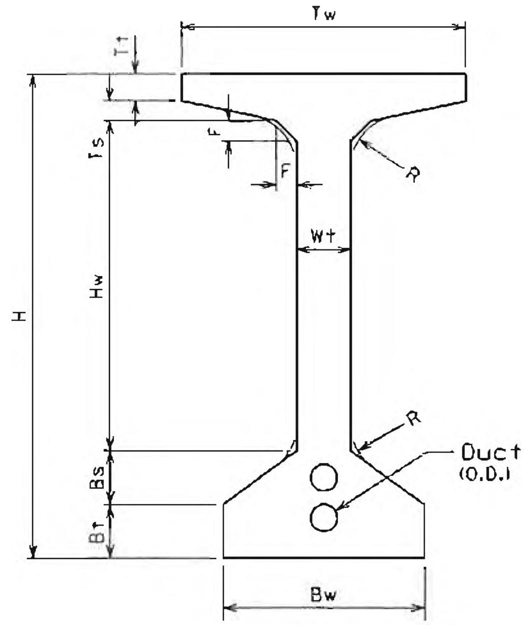
Table 4.1
General Loadings and Bridge Configuration for Comparison of Prestress vs. Post-Tensioning

Live Load	*Greater of: AASHTO HS-25 Truck Load or AASHTO HS-25 Lane Load*
Structural Deck Thickness	*7" (150 lbs/ft^3)*
Sacrificial Wearing Surface Thickness	*1" (150 lbs/ft^3)*
Haunch Thickness	*1" x top flange width (150 lbs/ft^3)*
Crossframes	*None – Assumed steel crossframes, weight is negligible*
Future Wearing Surface (Superimposed)	*25 psf (Distributed evenly to all beams)*
Parapet Load (Superimposed)	*1000 lbs/ft (Distributed evenly to all beams)*
Deck Concrete Strength	*4000 psi*
Beam Concrete Strength	*Concrete #1 – 4ksi @ Release, 6ksi @ Final* *Concrete #2 – 5ksi @ Release, 8ksi @ Final* *Concrete #3 – 6ksi @ Release, 10ksi @ Final*
Strand- Ultimate Stress	*0.5" dia., 7-wire strand, f's = 270 ksi (Initial Pull = 0.75*f's)*
Bridge Width – Out to Out of Deck	*48 ft.*
Bridge Width – Toe to Toe of Parapets	*45 ft. (Two parapets, 1.5 ft wide each)*
Number of Design Lanes	*3 Lanes*
Beam Spacing	*6 ft., 8 ft., 10 ft. and 12 ft.*

Prestressed I-beams were analyzed using a self-created spreadsheet following design examples from the PCI Bridge Design Manual (PCI, 1999). Designs involving post-tensioning were analyzed using CONSPLICE PT, Version 1.0 (LEAP 2001), a commercial design package developed and maintained by LEAP Software. This proprietary software is specifically used in designing post-tensioning and spliced girder bridges. CONSPLICE PT was also used to evaluate the lateral stability of all prestressed and post-tensioned beams during handling and transportation. AASHTO Standard Specifications (AASHTO, 1996) were utilized in all cases and ACI-209 (ACI, 1992) was utilized to model the effects of temperature, creep and shrinkage in all post-tensioned scenarios.

To demonstrate the benefits of post-tensioning and splicing, many of the most widely used 72"± deep I-beam precast sections used throughout the country were analyzed using post-tensioning and splicing. This depth was selected because it is commonly used, easily transported, fabrication forms require little modification, and nearly every state is familiar with its limits when used strictly as a prestressed I-beam. Included in Figure 4.1 are the beam sections and their properties of those investigated. Each precast beam section was analyzed with a given bridge configuration at various beam spacings to determine the maximum span of each precast section using prestressing and post-tensioning. Figure 4.2 depicts the various bridge configurations investigated.

In all, forty-four different scenarios were analyzed for both the prestressed and the post-tensioned cases. Table 4.2 provides the most pertinent information about each analysis including the maximum span, the area of prestressing used, the area of post-tensioning used, the number of post-tension tendons used and the lowest stability factors of safety for each beam during handling and transportation. For cases where three segments comprise the entire beam, the information given only refers to the middle segment, which was always the most critical segment. All graphical comparisons are derived from Table 4.2. A designer using any of the graphs produced in this thesis for a preliminary design should always bear in mind the design parameters stated in this text and the information provided in Figure 4.2. Variations in loading or dimensions can cause significant changes in the maximum span length. As with any design aid, good engineering judgment and an understanding of its derivation is essential to its proper use. These tables are only intended for preliminary use and should be verified using the local codes and procedures in a final design.



	H (in)	Tw (in)	Tt (in)	Ts (in)	Hw (in)	Bs (in)	Bt (in)	Bw (in)	F/R (in)	Wt (in)
OHIO MODIFIED 72"	72.00	36.00	4.00	2.00	49.00	9.00	8.00	26.00	3.00F	8.00
NEW ENGLAND BT 1800	70.87	47.24	3.35	1.97	52.95	3.94	8.66	31.89	7.87R	7.09
CALTRANS BT 1850	72.83	47.24	3.94	3.74	51.18	5.91	7.87	29.53	7.87R	7.87
AASHTO STD TYPE VI	72.00	42.00	5.00	3.00	46.00	10.00	8.00	28.00	4.00F	8.00
PCI BT 72"	72.00	42.00	3.50	2.00	56.00	4.50	6.00	26.00	2.00F	6.00
COLORADO -BT 72	72.00	43.00	3.00	2.00	54.00	6.00	6.50	26.97	2.00F	7.00
NEBRASKA NU 1800	70.88	48.02	2.56	1.75	50.25	5.50	5.31	39.38	7.89R	6.88
OHIO MODIFIED 60"	60.00	36.00	4.00	2.00	37.00	9.00	8.00	26.00	3.00F	8.00
OHIO MODIFIED 84"	84.00	36.00	4.00	2.00	61.00	9.00	8.00	26.00	3.00F	8.00

	Area (in^2)	Inertia (in^4)	Sb (in^3)	Yb (in)	St (in^3)	Yt (in)	Duct (in)
OHIO MODIFIED 72"	956	616,018	17,893	34.43	16,396	37.57	4.50
NEW ENGLAND BT 1800	958	655,855	19,490	33.65	17,621	37.22	3.94
CALTRANS BT 1850	1063	754,388	20,311	37.20	21,206	35.63	4.50
AASHTO STD TYPE VI	1085	733,320	20,168	36.36	20,576	35.64	4.50
PCI BT 72"	767	545,894	14,915	36.60	15,421	35.40	-
COLORADO -BT 72	864	594,937	16,634	35.77	16,421	36.23	3.94
NEBRASKA NU 1800	924	639,471	19,872	32.19	16,528	38.69	3.75
OHIO MODIFIED 60"	860	384,705	13,386	28.74	12,306	31.26	4.50
OHIO MODIFIED 84"	1052	916,011	22,809	40.16	20,894	43.48	4.50

FIGURE 4.1
Dimensions & Properties of Prestressed/ Post-Tensioned I-Beam Sections

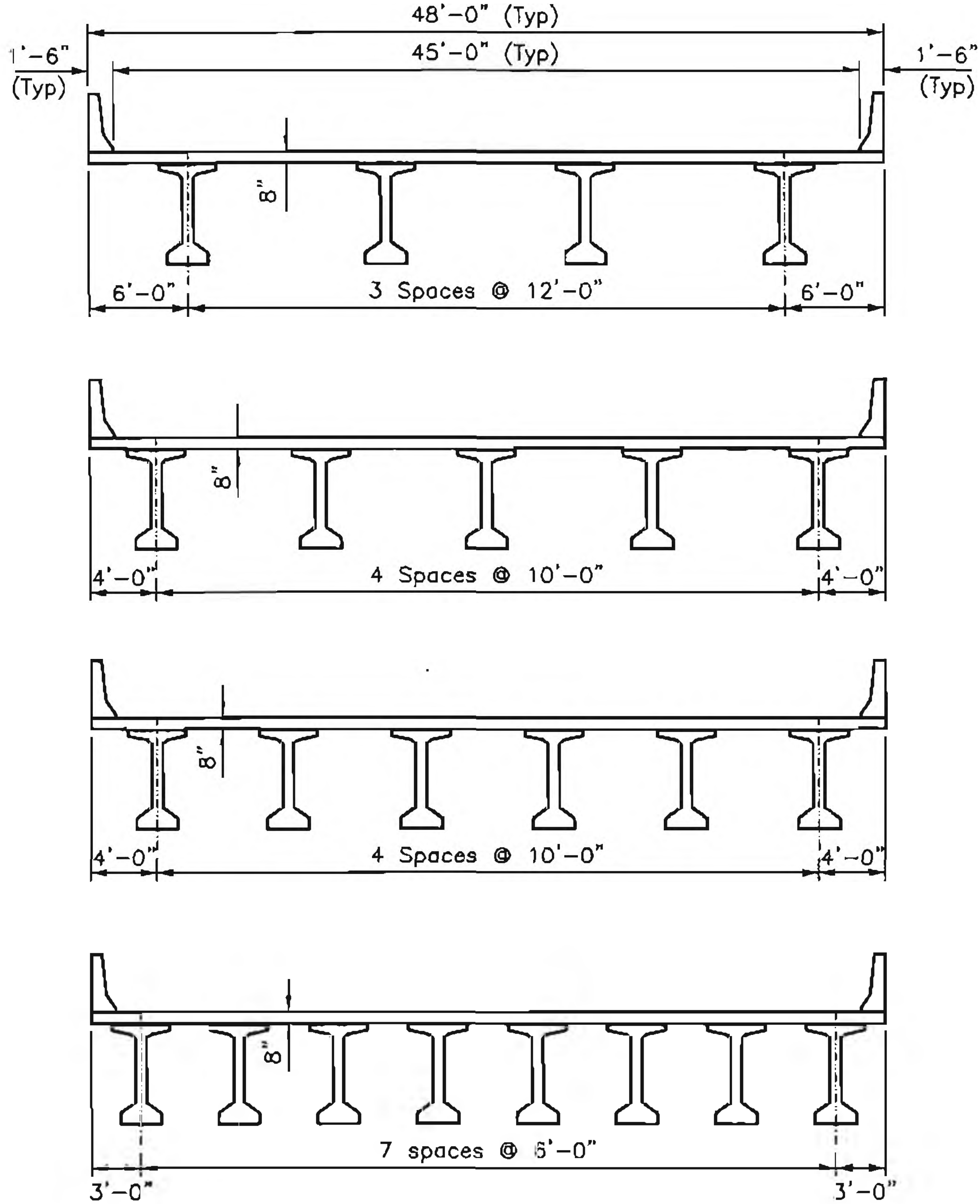


FIGURE 4.2
Various Bridge Configurations Investigated
12'-0", 10'-0", 8'-0" & 6'-0" Beam Spacings

TABLE 4.2
Prestressed and Post-Tensioned Beam Results

		6.00 Foot Beam Spacing					8.00 Foot Beam Spacing					10.00 Foot Beam Spacing					12.00 Foot Beam Spacing				
		Span	PS (in^2)	PT (in^2)	FS1	FS2	Span	PS (in^2)	PT (in^2)	FS1	FS2	Span	PS (in^2)	PT (in^2)	FS1	FS2	Span	PS (in^2)	PT (in^2)	FS1	FS2
Ohio 72" (R=4" F=6")	Prestress	139'	8.18		1.40	1.48	121'	7.35		1.97	1.81	107'	6.68		2.26	1.71	97'	8.35		2.59	1.71
	PT- 1 Seg											126'	4.34	1 @ 4.74	1.12	1.81	116'	4.01	1 @ 4.74	2.81	3.27
	PT- 3 Seg	157'	2.00	2 @ 4.74	6.39	5.78	150'	1.67	2 @ 4.74	7.60	7.38										
Ohio 72" (R=5" F=8")	Prestress	151'	10.35		1.20	1.39	132'	9.35		1.74	1.70	119'	8.68		1.98	1.65	107'	8.02		2.25	1.82
	PT- 1 Seg											158'	4.34	2 @ 4.74	1.23	1.98	143'	3.67	2 @ 4.74	1.72	2.21
	PT- 3 Seg	182'	5.34	2 @ 4.74	3.81	3.93	170'	4.68	2 @ 4.74	4.66	4.78	154'	3.01	2 @ 4.74	7.61	7.42	140'	3.01	2 @ 4.74	10.6	10.1
Ohio 72" (R=6" F=10")	Prestress	160'	12.86		1.10	1.47	143'	11.52		1.51	1.72	128'	10.69		1.94	1.94	115'	10.02		2.37	2.24
	PT- 1 Seg																162'	5.68	2 @ 4.74	1.17	1.78
	PT- 3 Seg	200'	7.68	2 @ 4.74	2.89	3.23	188'	7.68	2 @ 4.74	2.90	3.23	177'	7.68	2 @ 4.74	2.90	3.23					
New England BT 1800	Prestress	157'	11.02		1.73	2.28	137'	9.69		2.40	2.66	123'	9.02		3.06	3.14	111'	8.68		3.85	2.96
	PT- 1 Seg											165'	5.01	2 @ 3.98	1.69	2.54	150'	4.34	2 @ 3.98	2.32	3.17
	PT- 3 Seg	185'	5.68	2 @ 3.98	4.74	4.70	173'	5.34	2 @ 3.98	5.65	5.42										
CALTRANS BT 1850	Prestress	158'	11.36		1.50	1.64	139'	10.02		2.06	1.69	125'	9.35		2.59	2.29	112'	8.68		3.02	2.52
	PT- 1 Seg											168'	5.69	2 @ 4.74	1.47	2.28	154'	5.01	2 @ 4.74	1.99	2.73
	PT- 3 Seg	199'	6.68	2 @ 4.74	3.36	2.33	183'	6.01	2 @ 4.74	4.18	3.56										
AASHTO TYPE VI (72")	Prestress	158'	11.69		1.52	1.81	140'	10.52		2.08	2.15	125'	9.69		2.87	2.53	112'	8.85		3.07	2.80
	PT- 1 Seg											168'	6.35	2 @ 4.74	1.45	2.08	152'	5.34	2 @ 4.74	2.03	2.60
	PT- 3 Seg	196'	8.35	2 @ 4.74	3.47	3.23	185'	7.01	2 @ 4.74	4.44	3.94										
PCI BT 72"	Prestress	137'	7.01		1.78	2.16	117'	6.35		2.34	2.80	105'	6.01		2.45	2.78	93'	5.68		2.47	3.12
	PT- 1 Seg																				
	PT- 3 Seg																				
COLORADO BT 72"	Prestress	146'	8.68		1.51	1.70	126'	7.68		2.00	1.86	111'	7.01		2.34	2.08	100'	6.68		2.39	2.19
	PT- 1 Seg											148'	5.68	1 @ 3.98	1.65	2.20	130'	5.34	1 @ 3.98	2.34	3.45
	PT- 3 Seg	186'	4.01	2 @ 3.98	3.90	4.30	170'	3.34	2 @ 3.98	5.33	5.86										
NEBRASKA NU 1800P	Prestress	156'	10.02		2.05	2.98	136'	9.02		2.89	3.58	121'	8.35		3.75	4.29	110'	8.02		4.20	4.67
	PT- 1 Seg											154'	4.34	2 @ 3.37	2.45	3.48	139'	4.01	2 @ 3.37	3.38	4.26
	PT- 3 Seg	178'	4.68	2 @ 3.37	7.65	7.14	166'	4.68	2 @ 3.37	7.65	7.14										
OHIO 60"	Prestress	130'	8.85		1.55	2.94	113'	8.02		2.04	3.42	100'	7.35		2.27	3.72	90'	7.01		2.14	3.84
	PT- 1 Seg	150'	5.34	1 @ 4.74	1.22	3.02	138'	5.01	1 @ 4.74	1.55	3.46	121'	4.34	1 @ 4.74	2.32	4.56	108'	4.01	1 @ 4.74	3.12	5.55
	PT- 3 Seg																				
OHIO 84"	Prestress	170'	11.52		0.87	0.53	151'	10.52		1.23	0.57	136'	9.69		1.53	0.51	122'	9.02		1.78	0.39
	PT- 1 Seg																				
	PT- 3 Seg	201'	5.34	2 @ 4.74	2.94	1.78	184'	4.68	2 @ 4.74	3.89	2.44	170'	4.01	2 @ 4.74	5.86	4.04	156'	4.01	2 @ 4.74	5.87	4.04

FS1 = Factor of safety against failure during initial lifting out of forms. Concrete at release strength.
FS2 = Factor of safety against beam failure during truck transport. Concrete at final (28 day) strength.
PS = Total number of prestress strands used. Each strand area = 0.167 in^2
PT = Total number of post-tensioned strands used. Each strand area = 0.153 in^2

Graphical comparisons of the prestress maximum span lengths and the post-tensioned maximum span lengths are plotted vs. the beam spacing at which it is analyzed. These plots are presented in Figure 4.3. A side-by-side comparison of prestress vs. post-tensioning for each individual 72"± precast section is also provided at the end of this section. All 72"± sections are initially analyzed using the same concrete strength (Release = 5ksi and Final = 8ksi). This is referred to in the results as Concrete #2.

To compare the effects of varying the concrete strength, one "weaker" concrete strength was used (Release = 4ksi, Final = 6ksi) as well as one "stronger" concrete (Release = 6ksi, Final = 10ksi). These are referred to as Concrete #1 and Concrete #3 respectively. Analyses of the different concrete strengths are only performed on the Ohio 72" precast section. The results are shown in Figure 4.4. In summary, changing concrete strengths on a prestressed beam resulted in average maximum span change of 10 feet in either direction by decreasing (Concrete #1) or increasing (Concrete #3) the concrete strength. For the post-tensioned beam, the change in maximum span length was more pronounced. Decreasing the concrete strength (Concrete #1) resulted in a loss in the maximum span length of almost 25 feet and increasing the concrete strength (Concrete #3) resulted in an increase of 20 feet. Thus, concrete strength is an important factor for post-tensioned concrete beams. With some caution, one could extrapolate these results to other precast I-beam sections by comparing the relative performance of each beam section to those shown in Figure 4.3. For example, one might speculate that a New England BT 1800 spaced at 8 feet, might extend its maximum span length from 173 to 191 feet simply by increasing the concrete strength from Concrete #2 to Concrete #3. This 18-foot increase is inferred from the increase that the Ohio 72" beam experienced under these same parameters. In this manner, a designer has a reasonable starting point to begin his or her own analysis.

FIGURE 4.3
72" Prestressed and Post-Tensioned Sections, Concrete #2 (R=5ksi, F=8ksi)

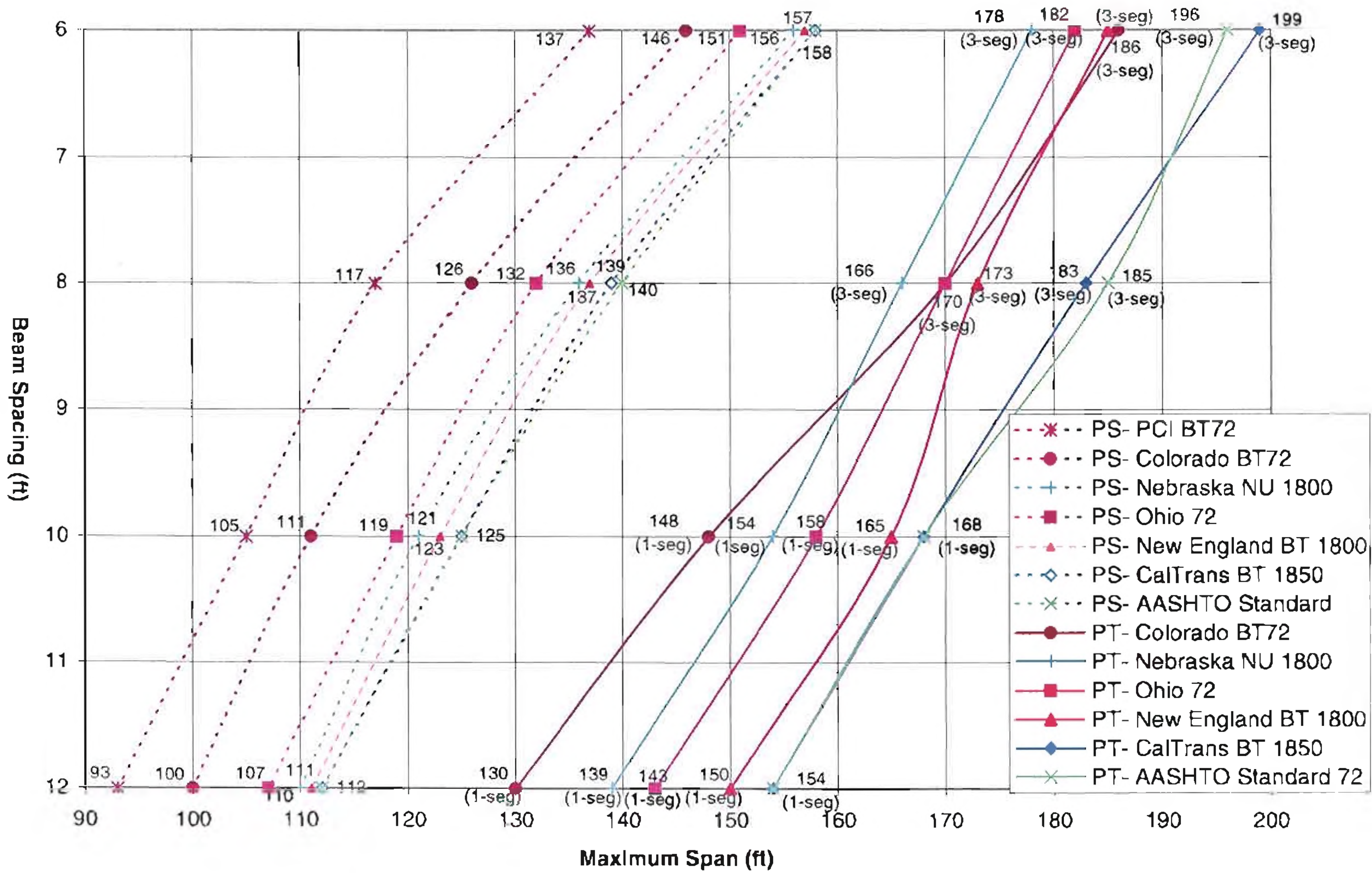


FIGURE 4.4
Ohio 72", Prestress vs. Post-Tensioning- Various Concrete Strengths

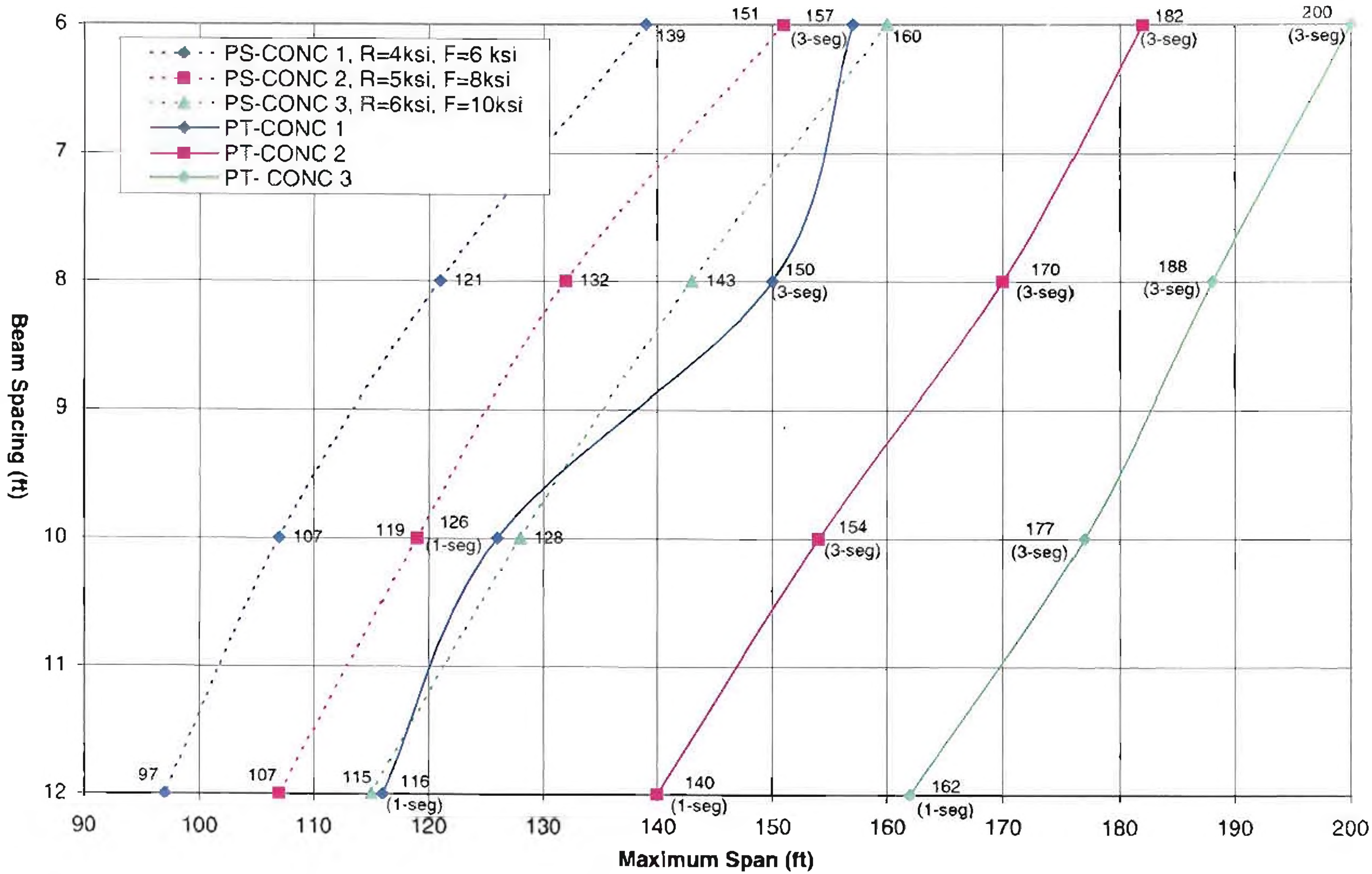
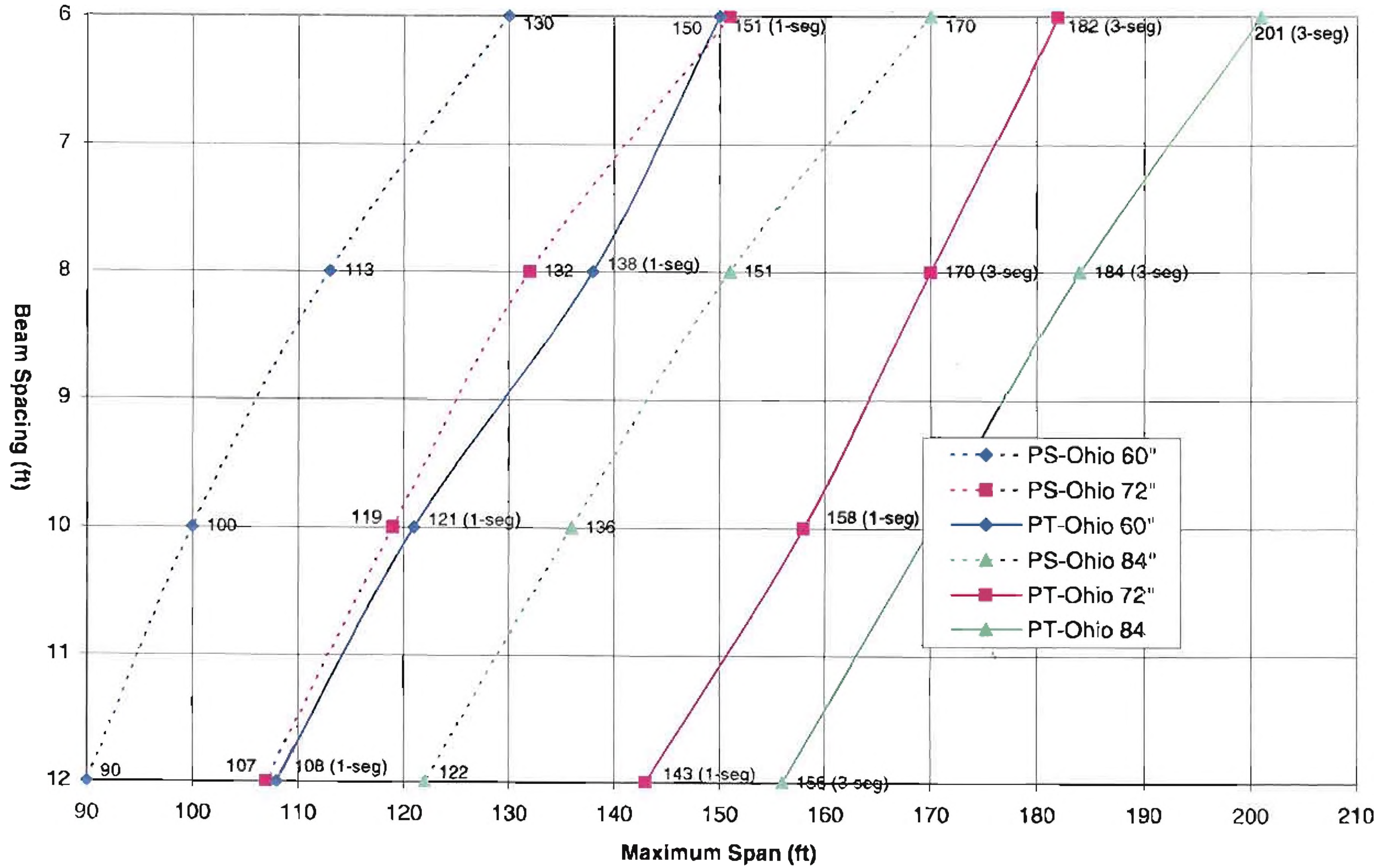


FIGURE 4.5
Ohio 60", 72" & 84" Prestress vs. Post-Tensioning (Concrete #2)



To compare the effects of varying the beam depth, one shallower and one deeper section are also analyzed to determine their maximum span lengths. All dimensions of the Ohio 72" beam flanges were held constant except the web height was decreased and increased by 12 inches. This results in the Ohio 60" section and the Ohio 84" section, respectively. Figure 4.5 compares the maximum span length of the three beam depths. For the prestressed I-beams, changing the beam depth resulted in increase or decrease in the maximum span length of approximately 18 feet. For the post-tensioned I-beam, decreasing the beam height by 12 inches resulted in a loss in the maximum span length of 35 feet. One reason for this substantial loss is that the Ohio 60" I-beam could only withstand a single post-tensioned duct. This is the result of the reduced area and section modulus. Increasing the beam height 12" to the Ohio 84" section only increased the maximum span length of the post-tensioned I-beam an average of 14 feet. This increase would have been much larger had a third post-tensioned duct been feasible, however the Ohio 84" beam sections could not tolerate this added force. It is likely that each beam series would be affected differently by increasing and decreasing the beam height and therefore, extrapolating these results to other beam series is not as directly applicable as the concrete strength comparison. The Ohio I-beam series has the smallest top flange of all the beam series investigated. It is therefore probable that other beam series with wider top flanges would allow another post-tensioned duct to be used resulting in substantially longer span lengths as the size of the top flange greatly influences the beam's section moduli.

C. Prestressed Concrete I-Beams

When a downward force is applied to a simply supported single span beam, the beam experiences compression in its top surface and tension in its bottom surface. Concrete is a material that resists compressive forces well, but is weak with respect to tensile forces. In most concrete structures, mild-reinforcing steel is used to resist the tensile forces. It is more effective, however, to add sufficient compression in the concrete beam by prestressing so that little or no tension is induced in the beam. Furthermore, adding the compressive force at a location below the centroid of the concrete beam induces additional compression in the bottom flange and tension in the top flange. This distance from the center of gravity of the beam to the center of gravity of the force is called the eccentricity. Theoretically, the compressive force applied with the right eccentricity can result in a beam that has almost no stress once the final loads are applied. This mechanism is accomplished via prestressing.

Prestressed beams are fabricated by tensioning several high tensile steel strands to approximately three-fourths of its ultimate capacity. Next, concrete is cast around the strands in the desired shape of the beam and cured to a predetermined strength. Once this strength is achieved, the forms are removed and the prestressing strands are cut beyond the ends of the beams. The cutting of the strands induces compression into the concrete immediately surrounding the strand along its bonded length.

Gravity and external loads result in tension and compression stresses that vary along the length of the beam. To control these stresses, strands are often deflected when initially stressed allowing deliberate manipulation of the stresses in the desired locations. Another effective method of controlling stresses is to debond strands for a prescribed length where a high prestresing force is not required. Debonding is a method of taking away the ability of the strand to transfer its force into the surrounding concrete. Debonding can be achieved by placing sheathing around the strand over the desired distance.

Precast concrete beams are available in various sizes and shapes depending upon their particular use. Bridge beams usually span large distances and carry heavy loads, so designers and owners try to utilize the most efficient section for design and fabrication. An efficient precast concrete I-beam section places the most mass at the top and bottom extremities of the beam while limiting the deadload of the beam itself. A wide top flange also facilitates a shorter design span for the deck that spans between the beams. Additionally, a wide bottom flange allows a greater number of prestressing strands to be used at a greater eccentricity. The wide flanges at the top and bottom also increase the lateral stability of the beam.

The design of a prestressed concrete I-beam requires the analysis of the stresses and moments of the beam not only once in place but also at the cutting or release of the strands. When the strands are released, the bottom of the beam is in compression. Once placed in the field and the dead loads and live loads are added, the compression at midspan of the beam in the bottom flange will be reduced, even to the point that the beam experiences a controlled amount of tension. The top flange will experience less compression at release and possibly even tension at the beam ends. As the loads are applied, the top flange at midspan will only increase in compression. The stresses at the midspan and at the endspan must be checked to ensure the allowable limits for tension and compression are not exceeded at any stage of the beam's life. When determining the maximum span for a given beam section in this research, it was always a matter of adding enough strands in the bulb to satisfy the required bottom tensile stresses in its final condition while not exceeding the allowable compressive strength in the bottom of the beam at release.

With the above case always being the limiting factor, the prestress case is rather predictable. When comparing the beam spacing to the maximum span length, fairly consistent curves were generated for every prestressed I-beam investigated. The beams also behaved reasonably proportionate to their respective moments of inertia. Maximum spans with a 12'-0" beam spacing ranged from 93 feet for the PCI 72" to112 feet for the AASHTO Type VI and the CalTrans 1850. By decreasing the spacing to 6'-0", the maximum span length of these same precast sections increase to 137 feet and 158 feet, respectively. These results will be compared later to those of the post-tensioned I-beams.

D. Single-Segment, Post-Tensioned Concrete I-Beams

Single-segment, post-tensioned bridge beams are constructed much like typical prestressed beams with a few exceptions. Upon fabrication of the beam, hollow ducts are cast into the beam, usually in a parabolic or draped arrangement. One or two ducts were generally found to provide the required capacity of the 72" deep I-beams explored in this thesis, but deeper sections and multiple span beams can have more. These ducts allow an external force to be applied to the beam after the concrete has reached the desired strength. Another general characteristic of post-tensioned beams is the presence of an end block at the beam-ends. An end block is typically as wide as the bottom flange and extends up to the bottom of the top flange. The length of the end block is based upon the amount of post-tensioning force that is to be applied, but typically does not exceed the depth of the beam.

Single-segment, post-tensioned beams are typically prestressed only enough to support their own self-weight and sometimes the weight of the wet concrete acting on the non-composite beam section. Post-tensioning forces are usually not applied until after the beams have been transported to the project site and erected into place. The post-tensioned forces are applied by tensioning strands located within the post-tensioning ducts. Multiple strands can share the same post-tensioning duct, sometimes as many as 30 or more. The strands are secured at one end with a piece of hardware called an anchor. One anchor per duct secures all the strands in the duct. A wedge shaped "chuck" takes advantage the force applied to grip the end of the strand and seat it into the anchor hardware. The strands can be pulled individually or a few at a time depending upon the capacity of the hydraulic jack used. A similar anchor system is used at the jacking end. All the strands within one duct are often referred to cumulatively as a single post-tensioned "tendon". Ducts are usually pressure grouted after stressing to prevent accumulation of water and corrosion of the strands.

Single-segment, post-tensioned concrete beam systems have both advantages and disadvantages. The major disadvantages are the complex fabrication of the end block and the extra construction steps in the field necessary to apply the post-tensioned force. Both require the fabricator and contractor to have technical expertise and equipment beyond that required for typical prestress beams. The construction schedule may need to be lengthened to allow time for both of these processes. Contractor and fabricator experience combined with proper planning may substantially lessen the extra time involved. The advantages gained by using post-tensioning can overcome the challenges listed above. Post-tensioning allows more resisting force to be applied to the beam at a lower center of gravity. In addition, this force is applied later in the life of the beam after it is removed from the forms thereby not delaying the productivity of the fabricator. These two factors allow precast beams to span further and/or reduce the number of beams fabricated and transported. These factors can result in an overall cost reduction for the project as a whole.

Conventional prestressed beams are generally limited to around 160-foot span lengths due to limitations in concrete strength, handling, erecting, and especially transportion of such lengths. Though single-segment, post-tensioned beams are less limited by concrete strength and handling constraints, transportation is still a governing issue. Based upon design capacity and stress calculations alone, single-segment, post-tensioned I-beams have approximately the same maximum span capabilities as three-segment, post-tensioned I-beams. As a result, single-segment, post-tensioned I-beams were not investigated for 6-foot and 8-foot beam spacings since in most cases the maximum span lengths achieved would have exceeded the maximum beam length that could be handled and transported. The particular advantage for the single-segment, post-tensioned beam construction evident from this research is that the beam spacing can be substantially increased for a given span length, thus eliminating the number of beams required.

For example, the maximum spans shown in Figure 4.4 indicate that an Ohio 72" prestressed beam (Concrete 3, PS = 12.859 in^2) spaced at 6 feet is capable of spanning 160 feet. Similarly, an Ohio 72" single-segment, post-tensioned beam (Concrete 3, 34 PS = 5.678 in^2, PT = 2 tendons @ 4.741 in^2) is capable of spanning 162 feet and can be spaced at 12 feet. Unfortunately, the factor of safety for handling is below the recommended 1.5 for both configurations at 1.10 and 1.17 respectively. Analyses of these same scenarios at 150 feet show the factor of safety increases to 1.33 and 1.52, respectively. Not only is safer handling achieved with post-tensioning, but only half as many beams are required. The reduction in the number of beams saves in fabrication, transportation and erection, which can result in overall project savings. These advantages may become even more evident on wide bridges, at bridge sites involving long transportation hauls or when multiple bridges are constructed by the same designer, fabricator and contractor team. Construction of multiple bridges using post-tensioning allow the processes to be refined and one-time costs to be lessened on a per beam basis for all parties involved.

The design of a single-segment, post-tensioned beam must consider several scenarios and time-dependent variables. Each design is checked at various stages of construction. For 72"± beam sections, two tendons are typically used. The first tendon is pulled prior to the placement of the deck, and therefore, acts on the non-composite beam section only. The second tendon is pulled after the deck has reached its desired strength, and therefore, acts on the composite section. This second tendon is commonly referred to as the liveload tendon as it gives the beam the added capacity to support superimposed deadloads and liveloads. The moment capacity, beam stresses, and deck stress must not only be checked for the final service conditions, but must also be checked at each stage of construction. Concrete decks typically have a shorter life cycle than the beams that support them. Ensuring that stresses in the beam will not be exceeded if the deck is removed and replaced sometime in the future is therefore important. Overstress of the beam is possible if a substantial amount of post-tensioning is applied to the composite beam and deck. The concern is that once the deck is removed, the high amount of prestress and post-tensioning may cause excessive compressive stresses in the bottom of the beam. Overstress when the bridge is redecked is often found to be a limiting case.

The ultimate goal of this thesis is to determine the maximum span lengths of the various beam sections. Additionally, evaluating how varying the concrete strength and beam depth affect the various types of precast construction investigated is also important. For the prestressed beam, the limiting factor of the beam length is the compressive stress in the bottom flange at release vs. the tensile stress in the bottom flange with all deadloads and liveloads applied. Strands are added to the prestressed beam from the bottom up until both limits are nearly reached. In the case of post-tensioning, prestress is added to the non-composite section and post-tensioning can be added to the non-composite section, the composite section, or both. Additionally, different amounts of each can be added at the various stages resulting in numerous possible scenarios. To determine which scenario yields the longest span, an Ohio 72″ beam is analyzed with many of these varied combinations. From these analyses, the following results were determined to be the most effective manner to maximize the span of a 72″± post-tensioned I-beam.

First, the use of two tendons yielded the best results. Given the characteristics of the section and material properties, using more than two tendons raises the post-tensioned center of gravity too high to be adequately affective considering the amount of stress (force/area) that is induced into the beam. One tendon does not utilize the full capacity of the section and the concrete while taking up too much area of what would be prestressing strands. Secondly, various trials show that to pull one tendon on the non-composite section and one on the composite section is better. A one time post-tension stressing operation is favorable for construction; however, the prestresss section alone with all available prestress strand locations occupied does not have the capacity to hold the weight of the "wet" non-composite concrete deck at the long lengths these beams can span if post-tensioned otherwise. Conversely, if both tendons are pulled on the non-composite section, excessive compressive stress in the bottom flange prematurely governs the maximum span length. Thirdly, it is typically the case that a minimum number of prestress strands, sufficient for transportation, is better in combination with the first tendon pull to prevent overstressing the bottom flange in compression. Lastly, for the purpose of maximizing the span, to maximize the number of post-tension strands per tendon and not "hold back" some capacity for a particular stage that may be exceeding stresses before others is generaly more advantageous. Adding or subtracting a few prestressing strands better accommodates adjustment of stresses for such purposes.

The limiting factor in prestressed beam designs is almost always accomplished by adding enough strands to satisfy Service 1 tensile stresses while not exceeding the compressive stress induced at release of the prestressing strands. For post-tensioned beams, it is not as simple, and no one rule always governed. The post-tensioning sequence requires stresses and moments to be checked at multiple times during construction and throughout the life of the beam. Different beam sections and even different beam spacings control at different stages in the design. Heavier sections were usually controlled by the compression in the redecking stage versus the allowable tension when all loads are present. The lighter sections, such as the Colorado 72" beam, often had little prestress initially. Therefore, tension in the bottom flange at release or compression in the top flange when the wet concrete is added had to be balanced vs the compression in the bottom flange at the redecking stage. In nearly all cases, redecking was one of the limiting stages. Typically, the heavier sections with a larger moment of inertia were easier to design. Not only were they able to handle the larger post-tensioning forces and span further, but adding or subtracting two prestress strands to fix an overstress in one stage did not dramatically change things in another. This was often the challenge with the lighter sections.

Web thickness is also an important characteristic of each post-tensioned beam section. Duct sizes were chosen that allowed for a minimum cover of 1.5" to be maintained on either side of the web over the duct. Assuming #4 bars are used for the vertical shear reinforcing, only 1" clearance is maintained over these bars. Though these clearances are less than that of a conventional prestressed beam, these clearances have been used in the past. The extra inch in duct size diameter that the tight clearance affords allows significantly more post-tensioning. For the PCI 72" beam, the web only allowed a 3" duct size. As a result of this small duct size, the absence of the prestressing strands in the web area and the low section modulus, the PCI 72" section did not perform well when post-tensioned. Though several analysis were attempted, the maximum span length for the PCI 72" section could not be increased using post-tensioning. Though there is no set rule, increasing the web thickness to a minimum of 8" for all post-tensioned sections would be beneficial. This would allow 1.75" cover over the duct (1.25" over a #4 stirrup) on either side of a 4.5" O.D. duct and would better accommodate the local flexure and shear stresses.

Furthermore, other scenarios were developed for different depths. To represent a shallower section, an Ohio 60" section was analyzed. For this section, a single tendon was found to be most effective. Two fully loaded tendons were beyond the capacity of the section and concrete strength; furthermore, applying the post-tensioning to the non-composite section was more effective than applying this force to the non-composite section. Though the advantage of applying the post-tensioned force to a composite section is lost, there is still an advantage to applying the external force to a much higher strength concrete than would be for prestress. Also, a significantly lower center of gravity for the post-tension force is maintained. These two characteristics alone allow the maximum span length of an Ohio 60" prestress beam to increase as much as 22% (25 feet). Similar to the Ohio 72" beam, the factor of safety for handling limits the maximum beam length for smaller spacings. For example, stress and moment calculations allow a maximum length of 150 feet for the Ohio 60" section, however the factor of safety for handling (1.22) is less than the allowable of 1.5. Conversely, handling of the Ohio 60" section at an 8 foot beam spacing (span = 138 feet) is within tolerable handling limits, as well as for larger beam spacings.

Increasing the depth of section was also investigated for the single-segment, post-tensioned beam. Unfortunately, increasing the depth does little to help the handling and transportation factor of safety for a given section. The maximum span length (143 feet) achieved by the Ohio 72" beam was re-analyzed with the Ohio 84" deep beam (both Concrete 2 and 12' spacing). Though design stresses decreased significantly, the handling factor of safety only increased from 1.72 to 1.78, and the transportation factor of safety decreased dramatically from 2.21 to 1.61. No further analysis was performed on the single-segment, post-tensioned beam using the Ohio 84" deep section as the predictably small increase in length will likely not prove cost effective. The Ohio 84" deep section, however, proves most beneficial when the three-segment, post-tensioned beam is investigated since transportation and handling problems are substantially reduced.

E. Three-Segment, Post-Tensioned Concrete I-Beams

Three-segment, post-tensioned bridge beams are constructed similar to a single-segment, post-tensioned beam except that that they are fabricated in three discrete sections and assembled at the site. Typically, this is accomplished by pouring a short cast-in-place section in the field between each segment to make one long beam. Three segments allow placement of the splice away from the areas of high moment, tension, and compression. Post-tensioning ducts must be positioned and aligned in such a way that each tendon can be tensioned over the entire length of the beam once the cast-in-place spliced sections have attained their required concrete strengths.

For a single span bridge, there are two possible procedures for assembling a three-segment, post-tensioned beam in the field. First, temporary bents are used to support the individual segments until the field splice is made. Once the connection is made and the cast-in-place section has achieved its required strength, then the first post-tensioning tendon may be pulled. Now that the beam has the capacity to support at least it's own dead load, the temporary bents may be disassembled. Using this arrangement, the beams must be accurately aligned in their final position. Fortunately, temporary or permanent crossframes are typically attached prior to this step assisting in the alignment. Utilizing this operation, the lifting crane's or cranes' capacity need only be sufficient to place the individual segments. The second method of installation is to make the connection at the site but on the ground near the bridge site. This may be necessary if temporary bents are not feasible. Once the connection is made on the ground and at least one post-tensioning tendon is pulled, the beam is moved into place; however, moving the entire beam as one segment requires special consideration. At its full length, the beam's weight is likely going to require at least two cranes. In addition, the lateral stability of the beam to be picked up must be analyzed as a whole to ensure it will not crack or fail in the lifting and moving process. This may require a third crane to support the beam at intermediate points or a steel stiffening truss be attached the beam during erection. Or, if this is known from the beginning that the second method must be used, the designer can intentionally select a beam section with very wide top and bottom flanges that offer sufficient lateral stability during erection. For the purpose of this thesis, temporary bents are assumed, and thus, the first construction method described was used.

There are various ways to splice the segments together in the field. One of the most common techniques is to cast a short 1-ft to 3-ft section in the field. In this cast-in-place section, mild reinforcing and/or prestressing strands, extending from each end of the individual segments, are lap spliced, mechanically spliced, or welded to reinforce this section and provide continuity. Localized post-tensioning is also sometimes used for these purposes. Beam-ends are often designed to be roughened or cast irregular to facilitate shear transfer. The placement of the splice along the beam length is usually around the end quarter points of the beam. Pushing the splice out even further to approximately 20 percent of the beam's total length allows more prestressing strands to be placed in the middle section of the beam, while still enabling the middle section to be laterally stable and easily transported.

Once the three individual pieces are fabricated, and the cast-in-place field splices are made, the remainder of the construction sequence is identical to that of the single-segment, post-tensioned beams. The characteristics and design of the spliced beams are, therefore, similar to that of the one-piece post-tensioned beams. An important comparison is to determine whether a one segment, post-tensioned beam or a three-segment, post-tensioned beam can span further with all other things held constant, regardless of transportation issues. For this comparison, an Ohio 72" I-beam with concrete strengths of 5ksi at release and 8ksi (Concrete #2) at final was analyzed at 10 foot and 12 foot beam spacings. It was found that the maximum span differs only one to three feet. As a result, the choice to use a single-segment versus a three-segment should not be based upon which method allows the beam to span further, but should be based upon the lateral stability and ease of transportation of the beam to the site. Consequently, three-segment beams were only analyzed for beam spacings that could not be transported in a singled segment. The 72"± three-segment, post-tensioned beams were analyzed at 6'-0" and 8'-0" spacings. Graphically, one-segment and three-segment results were combined to represent a maximum span length curve for post-tensioning in general.

Similar to one-segment, post-tensioned beams, two post-tensioning tendons were developed to find the maximum span length of the three-segment beams. Similar to the one-segment, post-tensioned beams, the largest possible post-tensioning ducts were used to their full capacity. Prestressing strands were utilized in all three segments sufficient for transportation as a minimum. Additional prestressing strands were added to the middle segment to supplement the post-tensioning and extend the maximum span length. The short end spans typically only required 6 to 8 prestressing strands to facilitate transportation of the beams to the site. The splice itself was not designed as a part of this thesis. Considering the location of the splice far from the midspan of the beam, splice details would not be limiting factor and would have little impact on the maximum attainable span length. Lateral stability or transportation was never an issue for the three-segment, post-tensioned beams, as the maximum middle segment never exceeded 125 feet.

The three-segment analysis showed that 72"± I-beams typically used for prestressing could be used to span upwards of 180 feet using Concrete #2 (5ksi-Rel & 8ksi-Fin). This same concrete allowed two sections, the Standard AASHTO 72" beam and the CalTrans 1850, to span over 195 feet. In order to determine the effect of different strength concrete used with post-tensioning, the Ohio 72" beam was analyzed using the three different concrete strengths. This comparison shows that the Ohio 72" beam at a 6 foot spacing could be increased from 182 feet to 200 feet by increasing the concrete strengths from concrete #2 (5ksi-Rel & 8ksi-Fin) to Concrete #3 (6ksi-Rel & 10ksi-Fin). Other beams with a greater moment of inertia would likely span further. Similarly, increasing the depth of the beam from 72" to 84" allows the maximum span to increase from 182 feet to 201 feet, both of which used Concrete #2.

The sections analyzed in this paper were unchanged from their typical published dimensions. In some cases, changing selected dimensions if local fabricators are consulted might be advantageous. Increasing nominal web thickness as little as an inch may allow larger duct sizes for some of the thinner webbed sections. Some fabricators can accomplish this by holding the forms apart 1 inch if the bottom gap can be sealed off. This also has the added benefit of increasing the section properties. Wider top flanges may solve transportation issues. Thickening the top flange as little as an inch may be enough increase in the section properties to allow a little more length of span if necessary. These items should be discussed at length with local fabricators and contractors to assess their capabilities and limitations in changing section dimensions. Contractors and fabricators can also provide other valuable insight and suggestions that will result in more constructable and economical finished products.

FIGURE 4.6
Colorado 72", Prestress vs. Post-Tensioning (Concrete #2)

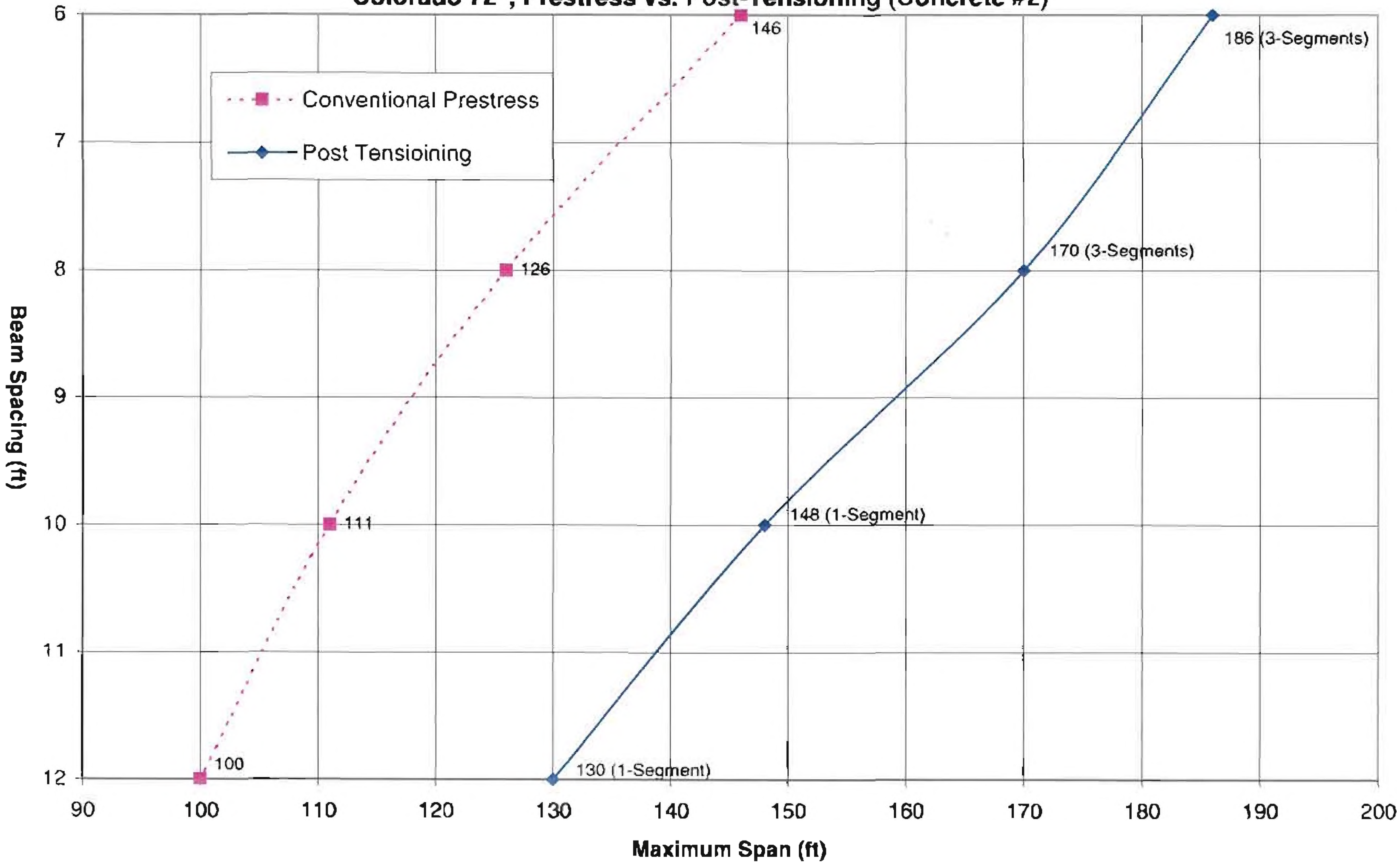


FIGURE 4.7
NU1800P, Prestress vs. Post-Tensioning (Concrete #2)

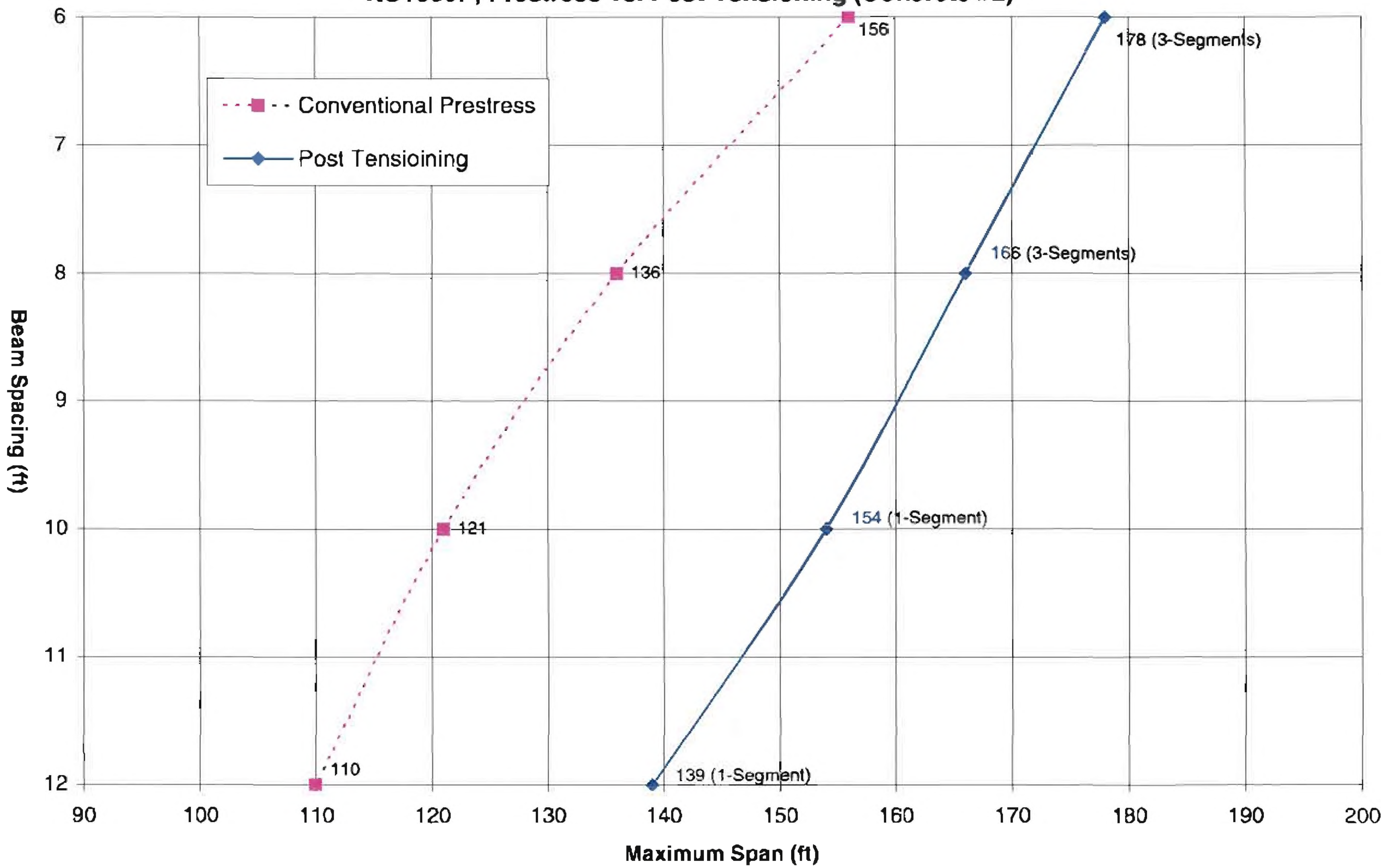


FIGURE 4.8

Ohio 72", Prestress vs. Post-Tensioning (Concrete #2)

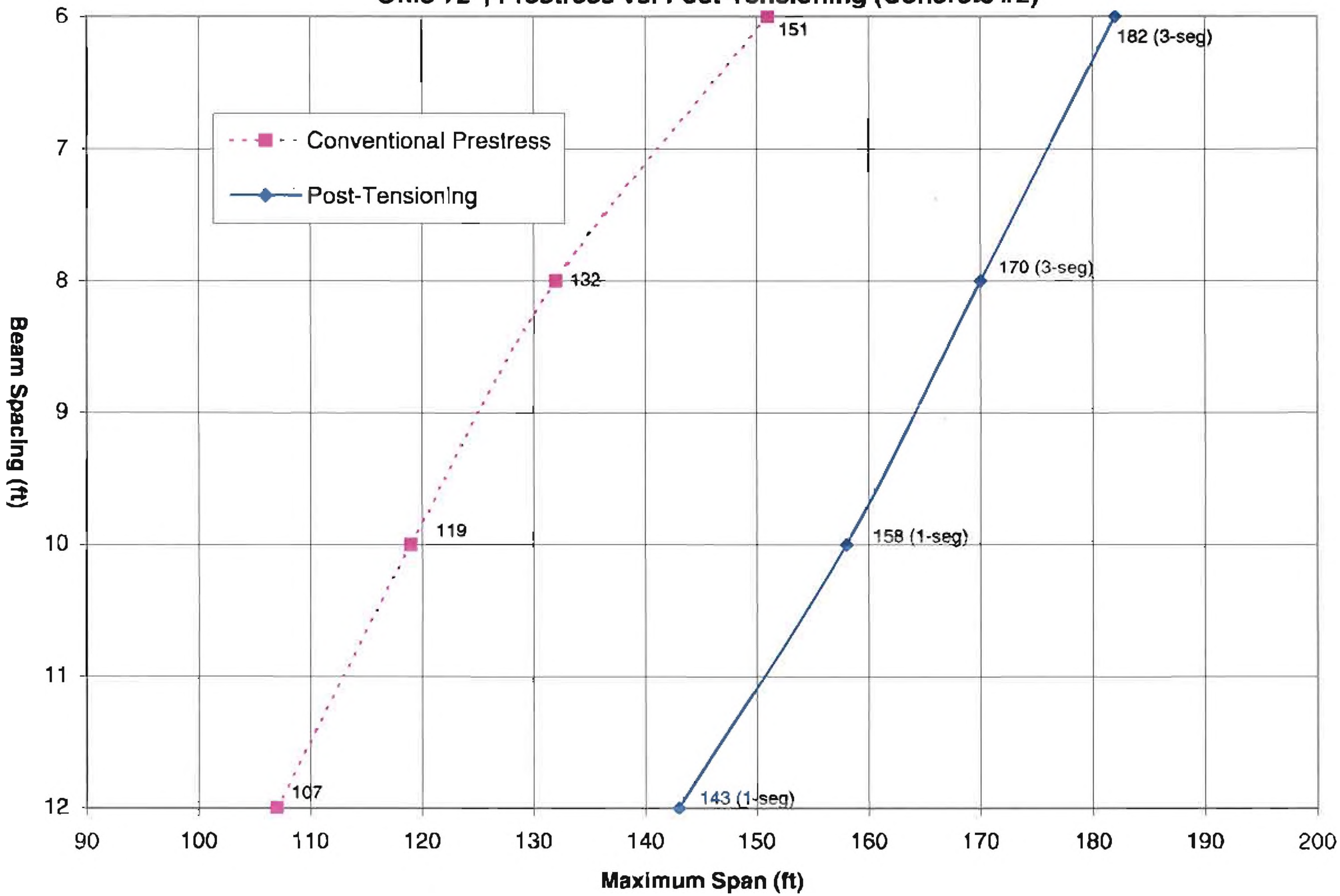


FIGURE 4.9
New England BT 1800, Prestress vs. Post-Tensioning (Concrete #2)

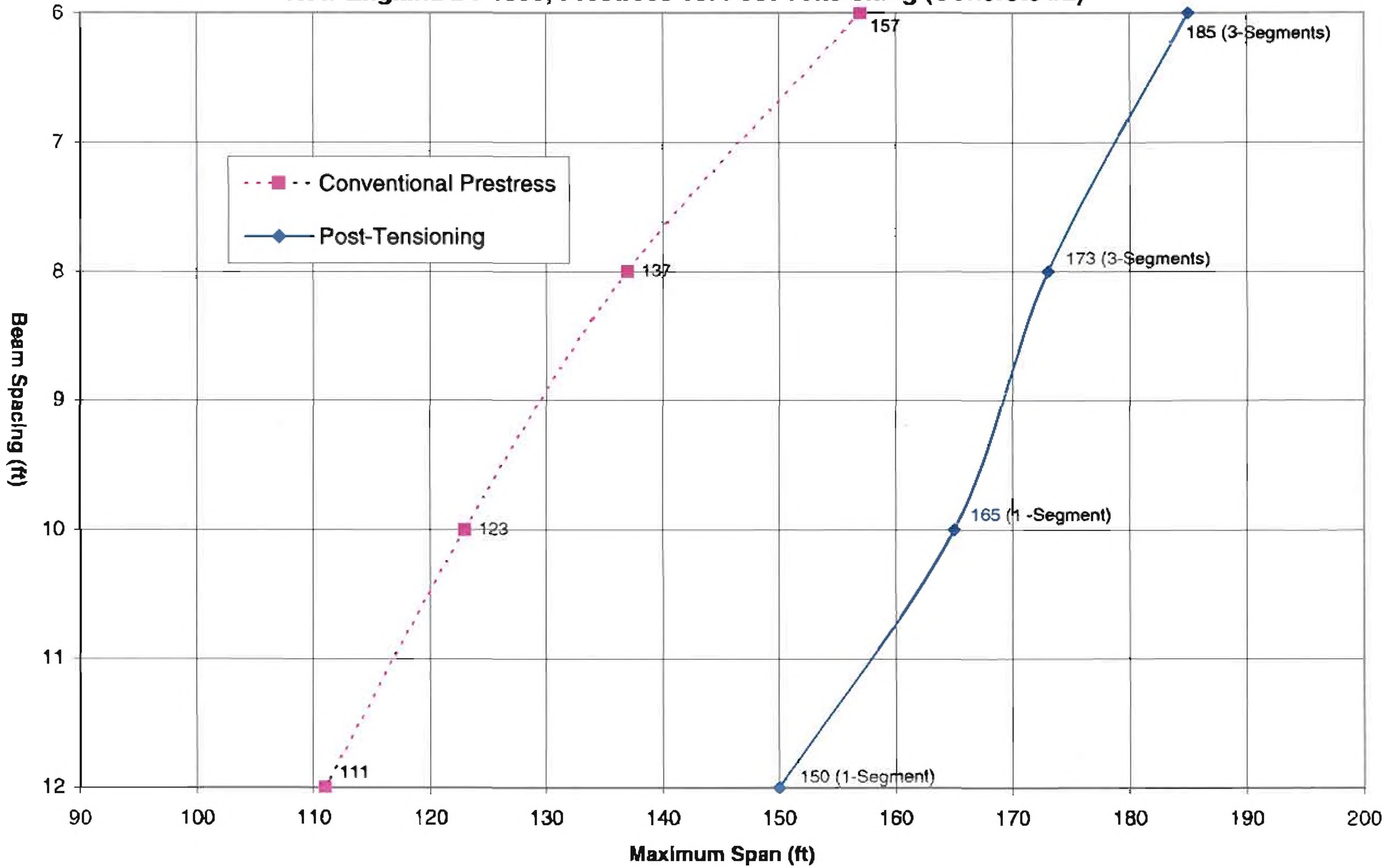


FIGURE 4.10
CalTrans BT 1850, Prestress vs. Post-Tensioning (Concrete #2)

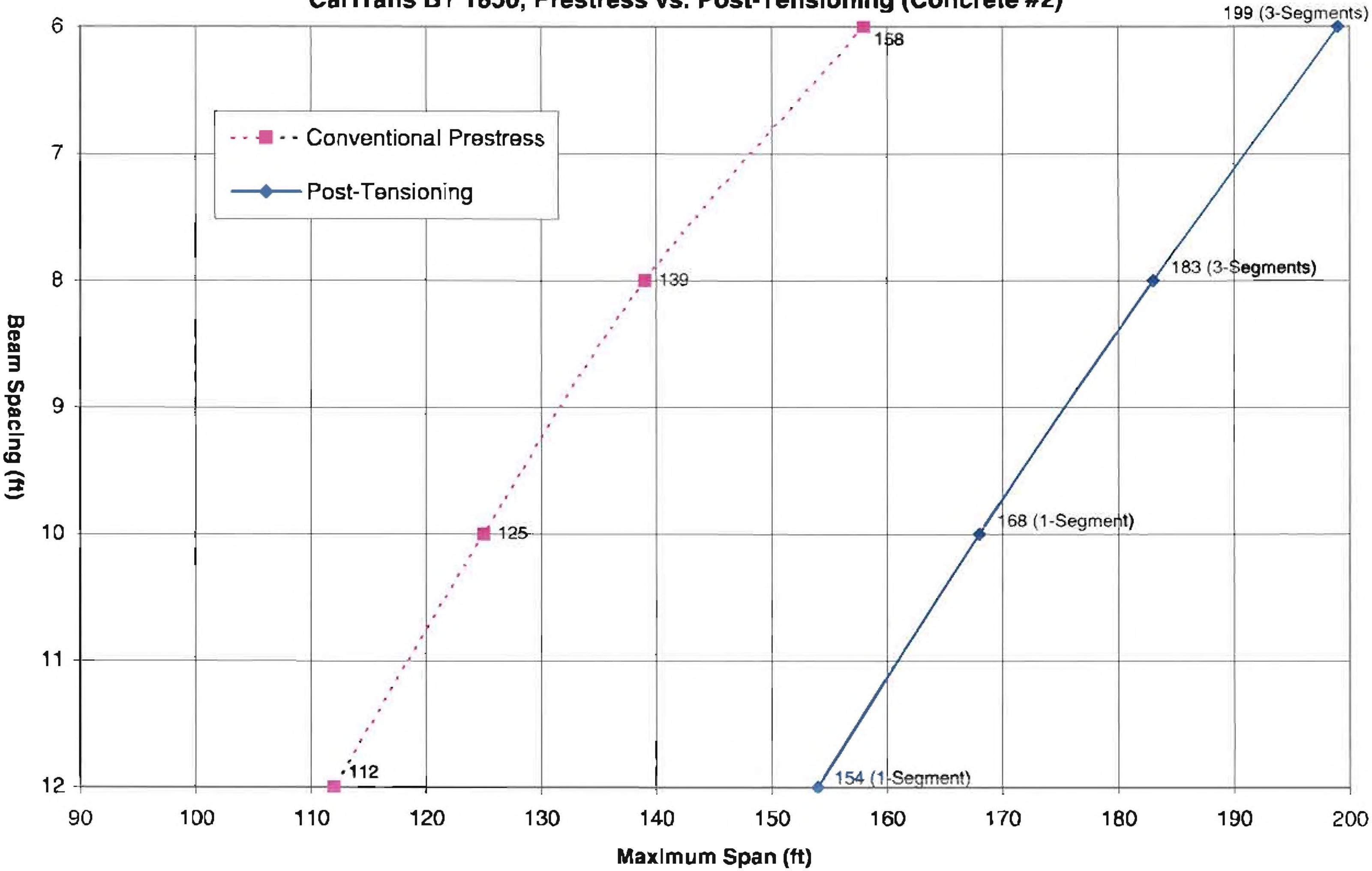
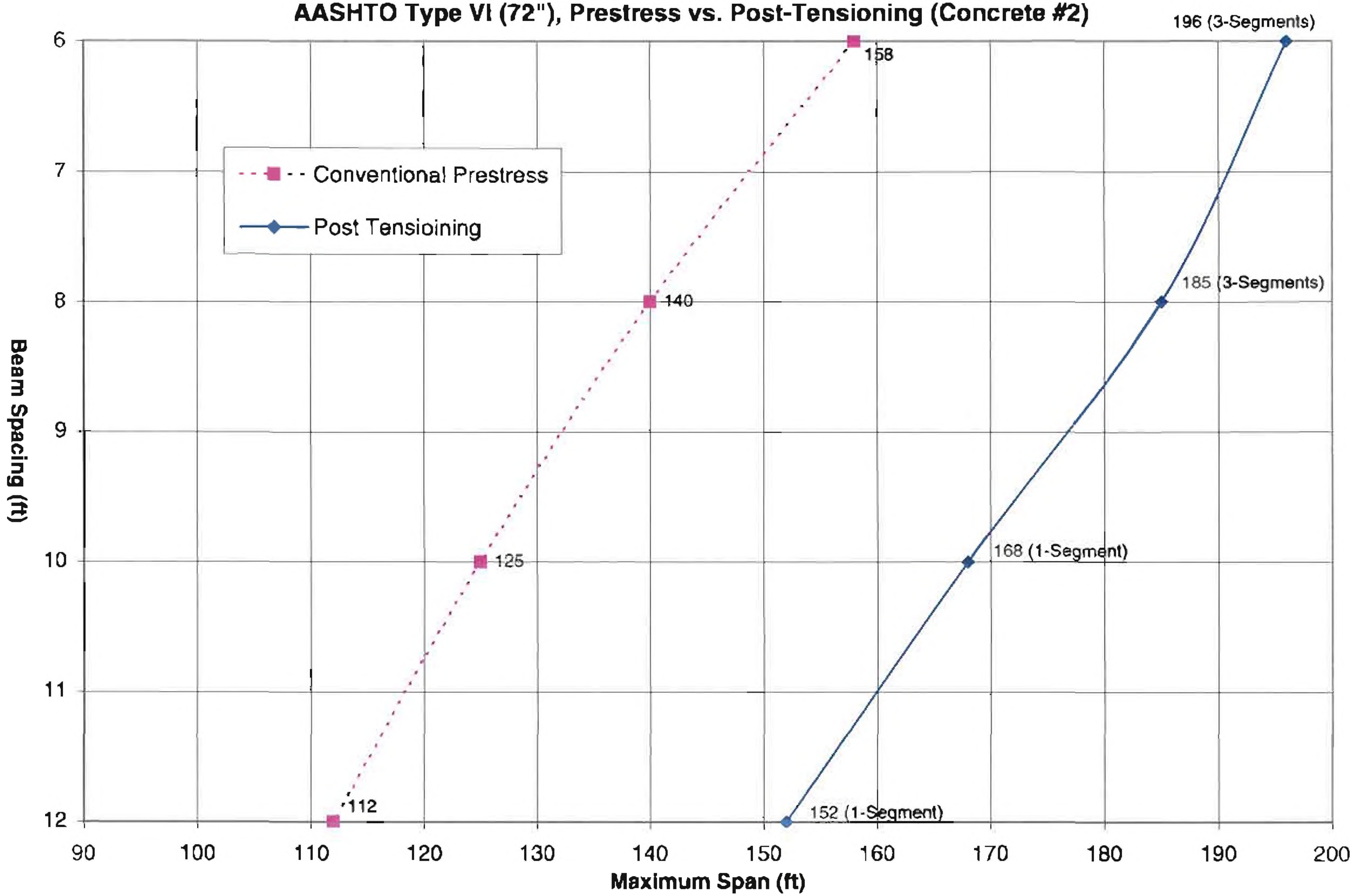


FIGURE 4.11
AASHTO Type VI (72"), Prestress vs. Post-Tensioning (Concrete #2)



CHAPTER V

CONCLUSIONS

The analyses in this thesis demonstrate that post-tensioning can be beneficial and have certain advantageous in comparison to prestressing. The following advantages were found regarding 72″ I-beams:

- Post-tensioning of single span precast I-beam bridges can extend the maximum span length by as much as 35% adding more than 40 ft of length to a span.
- Span lengths up to 200 feet for a three-segment, post-tensioned I-beam can be achieved with the combination high strength concrete (10ksi) and a 6-foot beam spacing.
- Half the number of beams can be eliminated by doubling the center-to-center beam spacing in comparison to prestress.
- Increasing the beam depth by 12 inches (84″ deep beam) extends the maximum span length approximately 20 feet for post-tensioning.
- The lateral stability for handling and transportation is improved through the use of post-tensioning, especially when three segments are used.

Additionally, through the process of analyzing many different scenarios, it is possible to develop some general guidelines to aid other designers in maximizing span lengths of post-tensioned I-beams. Though the analyses performed in this thesis are primarily based on 72" I-beams, many guidelines are applicable to other situations. These guidelines are as follows:

- Use the largest diameter post-tensioning duct size allowable without violating the necessary cover over the reinforcing steel and duct in the web of the I-beam. Thicker webs allow larger ducts, and therefore, more post-tensioning.
- For a 72" deep I-beam, two ducts are generally sufficient to maximize the span capabilities of the I-beam section. Additionally, the lower tendon pulled on the non-composite section and the upper tendon pulled on the composite section is generally most beneficial.
- Removal of the deck for a future deck replacement is often a governing load case and should be considered.
- As with prestressed I-beams, large beam sections with wide top and bottom flanges allow the furthest spans. The wide flanges also improve lateral stability.
- High concrete strengths are recommended, especially final concrete strengths of 8 ksi or more.
- Locating the splice of three-segment, post-tensioned I-beams approximately 20% from either end greatly reduces the moment stresses in the splice.
- Adding additional strands in the middle section of a three-segment, post-tensioned I-beam adds significant capacity to the beam.

Post-tensioning and splicing techniques described in this thesis extend the maximum span range of precast allowing it to be used in many situations it would not otherwise have been considered. The use of post-tensioning and splicing will likely continue to grow as owners, designers, contractors, and fabricators become more familiar with post-tensioning techniques, uses, and advantages. Designers and owners are the key to pushing the use of post-tensioning and splicing in bridge structures as these two entities determine the vast majority of bridge types. Through continued research, experimentation, and use, post-tensioning and splicing precast concrete beams in bridge applications may one day become common practice.

CHAPTER VI

RECCOMMENDATIONS

Only a limited range of I-beam sections have been considered to determine the advantages and practices of using post-tensioning for single span bridges. In addition to different beam depths, various precast sections such as precast slabs, boxbeams and "U-beams" may benefit from these types of construction as well. Future studies involving these sections would also aid in the development of this technology and promoting more widespread use

Multi-span spliced and post-tensioned precast beams are yet another area that future studies would prove beneficial with a wide array of topics. A study on multi-span post-tensioned precast beams should consider both constant depth and variable depth girders and the economics that can be gained by each. Analysis of multi-span construction should also address the construction techniques used to support the beam until the splice is made such as strongbacks, Cazaly hangers, temporary supports and/or fixed piers.

Further studies could concentrate on the endblocks and/or splices of the post-tensioned precast beams. The end anchorages of post-tensioned bridge beams are subjected to large compressive point loads. Proper distribution of these forces from the point of anchorage to the rest of the beam is essential to prevent cracking or spalling of the beam, which could lead to a significant loss in the post-tensioned force. Further studies on the splice of a post-tensioned beam could help aid in developing standard details and design procedures for this connection. Spliced mild reinforcing, spliced prestressing strands, and local post-tensioning have all been used in the past in conjunction with the normal longitudinal post-tensioning. Establishing the effectiveness of each splice and how it affects the overall model should be explored in more detail. Though a finite element analysis is one procedure of doing so, the best procedure would be to test and instrument a full-scale model. This could be done in the laboratory or in the field. Though the cost of testing a full-scale model is substantial, much could be learned about end anchorages, splices, temperature and shrinkage effects, and how well the models truly replicate the stresses of post-tensioned bridge beam in general.

Design software, design guides and codes will also help facilitate the growth of this technology. Little guidance on this subject matter is available in the United States. Two such studies currently in progress include the Chapter 11 of the PCI Bridge Design Manual [9] and the NCHRP 12-57 Report "Extending Span Ranges of Precast, Prestressed Concrete Girders".

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APPENDIX A

State DOT
Sample Questionnaire

Spliced Prestress/Post-tensioned Concrete Bridge Questionnaire

1. State____________, County______________, Owner_______________________________

2. Circle Deliver Process: a. Design/bid/build. b. Design/Build c. Value Engineering

3. Designed By: __________________ 4. Months to complete______5. Year completed_______

6. Constructed By: ________________7. Months to complete______ 8. Year completed_______

9. Beams Fabricated By: _____________10. Beams transported by: a. Truck b. Barge c. Other

11. Route _______________________________ 12. over ______________________________

13. Number of spans_______ 14. Span lengths____________________15. Tallest pier _______

16. Number of lanes_________ 19.Total deck width____________ 20. Beam Spacing_________

21. Live Load Design _______________22. Future Wearing Surface Design Load________(psf)

23. Deck type________________24. Deck thickness_________ 25. Deck Conc. (f'c)__________

Questions 26 thru 41 all pertain to the beam section at midspan of the longest span:

26. Depth of beam _______27. Top flange width___________28. Bottom flange width________

29. Web thickness________ Beam concrete strength: 30. Release_______31. Final(28 day)_____

32. Number of post-tensioned strands______ 33. Size and type of PT strands________________

34. Duct Size______35. Draped or straight PT strands_______ 36. Pull-Force per Strand_______

37. Number of prestressed strands_________ 38. Size and type of PS strands________________

40. Draped or straight PS strands ______________ 41. Pull-force per PS strand______________

Questions 42 thru 56 all pertain to the beam section over the pier adjacent to the longest span:

42. Depth of beam _______43. Top flange width___________44. Bottom flange width________

45. Web thickness________ Beam concrete strength: 46. Release_______47. Final(28 day)_____

48. Number of post-tensioned strands______ 49. Size and type of PT strands________________

50. Duct Size______51. Draped or straight PT strands_______ 52. Pull-Force per Strand_______

53. Number of prestressed strands_________ 54. Size and type of PS strands________________

55. Draped or straight PS strands ______________ 56. Pull-force per PS strand______________

57. Type of Splice Method Used: *(Circle One)*

a. Reinforced splice: Mild reinforcing and/or strands extending out of the ends of the beams are supported temporarily and cast together.

b. Cast-in-place, post-tensioned splice: beams are temporarily supported while cast-in-place concrete is poured. Post-tensioning is applied after the concrete joint has attained the required strength.

c. Stitched Splice: Strands or threaded bars extending from ends of prestressed beams are clamped or spiced together and a cast-in-place concrete joint is poured.

d. Drop-in-splice, hinged: The drop-in segment uses a permanent mechanical hinge.

e. Drop-in-splice, post-tensioned: The drop-in segment is immediately post-tensioned locally to induce contunity.

f. Structural Steel Splice: Overlapping steel plates are cast in the end s and bolted or welded together then encased in concrete.

g. Epoxy-filled post-tensioned splice: The gap between mated or match-cast beam ends are filled with epoxy get or grout. After initial set, post-tensioning is applied.

58. Were temporary supports require while connections were made? (yes or no)______________.

59. Was the connection made on the ground or in it's final position? (Ground or Final)__________.

60. How many cranes were used?__________ 61. What size crane(s) were used?____________.

61. Overall bid construction cost____________ 62. Actual cost of construction_____________.

63. What other bridge types were considered. Provide reasons not used. (Provide estimated costs if possible. ____

____

____

____

64. Please provide any additional comments or elaboration on the above questions concerning the design or construction of the bridge. This may relate to concerning any problems encountered, better suggestions, innovations of the bridges serviceability since it's completion that you may feel are relative to this research.

____

____

____

____

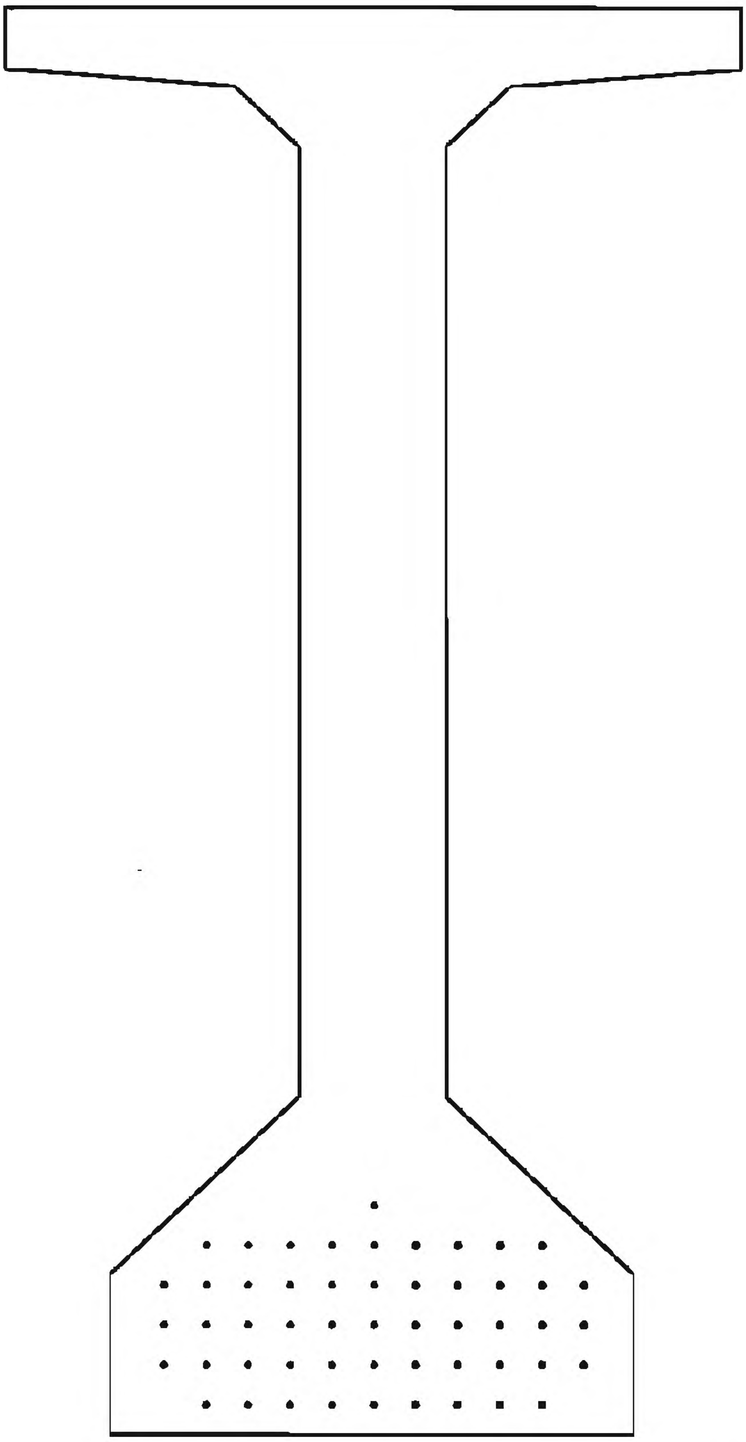
____

APPENDIX B

Prestressed I-beam
Spreadsheet

Design Example

Beam at Midspan

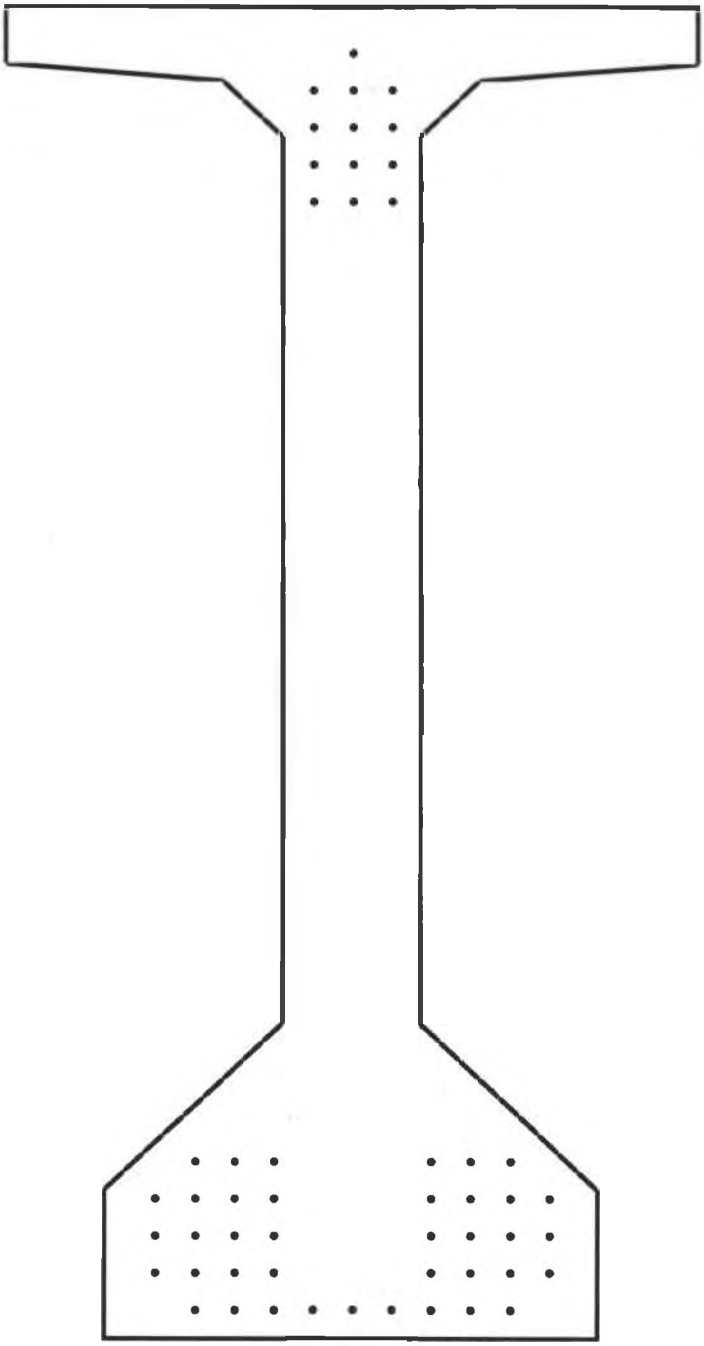


Dist. From Bottom, y_b	Number of Strands	Tot. Strand Area (in^2)	A*y_b
72 in	0	0	0
71 in	0	0	0
70 in	0	0	0
69 in	0	0	0
68 in	0	0	0
67 in	0	0	0
66 in	0	0	0
65 in	0	0	0
64 in	0	0	0
63 in	0	0	0
62 in	0	0	0
61 in	0	0	0
60 in	0	0	0
59 in	0	0	0
58 in	0	0	0
57 in	0	0	0
56 in	0	0	0
55 in	0	0	0
54 in	0	0	0
53 in	0	0	0
52 in	0	0	0
51 in	0	0	0
50 in	0	0	0
49 in	0	0	0
48 in	0	0	0
47 in	0	0	0
46 in	0	0	0
45 in	0	0	0
44 in	0	0	0
43 in	0	0	0
42 in	0	0	0
41 in	0	0	0
40 in	0	0	0
39 in	0	0	0
38 in	0	0	0
37 in	0	0	0
36 in	0	0	0
35 in	0	0	0
34 in	0	0	0
33 in	0	0	0
32 in	0	0	0
31 in	0	0	0
30 in	0	0	0
29 in	0	0	0
28 in	0	0	0
27 in	0	0	0
26 in	0	0	0
25 in	0	0	0
24 in	0	0	0
23 in	0	0	0
22 in	0	0	0
21 in	0	0	0
20 in	0	0	0
19 in	0	0	0
18 in	0	0	0
17 in	0	0	0
16 in	0	0	0
15 in	0	0	0
14 in	0	0	0
13 in	0	0	0
12 in	1	0.167	2.004
11 in	0	0	0
10 in	9	1.503	15.03
9 in	0	0	0
8 in	11	1.837	14.696
7 in	0	0	0
6 in	11	1.837	11.022
5 in	0	0	0
4 in	11	1.837	7.348
3 in	0	0	0
2 in	9	1.503	3.006
1 in	0	0	0
Σ =	52	8.684	53.106

C.G. of strands @ midspan = y_{bs} = 6.12 in

eccentricity = yb-ybs = 28.31 in

Beam at End



Dist. From Bottom	Number of Strands	(w/o debond) Tot. Strand Area (in²)	A^*y_b	Number of of Draped Strands
72 in	0	0	0	0
71 in	0	0	0	0
70 in	1	0.167	11.69	1
69 in	0	0	0	0
68 in	3	0.501	34.068	3
67 in	0	0	0	0
66 in	3	0.501	33.066	3
65 in	0	0	0	0
64 in	3	0.501	32.064	3
63 in	0	0	0	0
62 in	3	0.501	31.062	3
61 in	0	0	0	0
60 in	0	0	0	0
59 in	0	0	0	0
58 in	0	0	0	0
57 in	0	0	0	0
56 in	0	0	0	0
55 in	0	0	0	0
54 in	0	0	0	0
53 in	0	0	0	0
52 in	0	0	0	0
51 in	0	0	0	0
50 in	0	0	0	0
49 in	0	0	0	0
48 in	0	0	0	0
47 in	0	0	0	0
46 in	0	0	0	0
45 in	0	0	0	0
44 in	0	0	0	0
43 in	0	0	0	0
42 in	0	0	0	0
41 in	0	0	0	0
40 in	0	0	0	0
39 in	0	0	0	0
38 in	0	0	0	0
37 in	0	0	0	0
36 in	0	0	0	0
35 in	0	0	0	0
34 in	0	0	0	0
33 in	0	0	0	0
32 in	0	0	0	0
31 in	0	0	0	0
30 in	0	0	0	0
29 in	0	0	0	0
28 in	0	0	0	0
27 in	0	0	0	0
26 in	0	0	0	0
25 in	0	0	0	0
24 in	0	0	0	0
23 in	0	0	0	0
22 in	0	0	0	0
21 in	0	0	0	0
20 in	0	0	0	0
19 in	0	0	0	0
18 in	0	0	0	0
17 in	0	0	0	0
16 in	0	0	0	0
15 in	0	0	0	0
14 in	0	0	0	0
13 in	0	0	0	0
12 in	0	0	0	0
11 in	0	0	0	0
10 in	6	1.002	10.02	0
9 in	0	0	0	0
8 in	8	1.336	10.688	0
7 in	0	0	0	0
6 in	8	1.336	8.016	0
5 in	0	0	0	0
4 in	8	1.336	5.344	0
3 in	0	0	0	0
2 in	9	1.503	3.006	0
1 in	0	0	0	0
Σ =	52	8.684	179.02	13

C.G. of strands @ end of beam (w/o debond) $= y_b =$ 20.62 in
eccentricity (w/o debond) = yb-ybs = 13.81 in

Group 1 Debond							
# strands debonded =	2	AVG. Dist from bot =	2	in	Length of Debond =	6.00	ft

Group 2 Debond							
# strands debonded =	2	AVG. Dist from bot =	2	in	Length of Debond =	3.00	ft

BRIDGE INFO

Total width of Bridge =	48.00	ft.
Total width of Both Parapets =	3.00	ft.
Toe to Toe of Parapets =	45.00	ft.
Span Length C/C Bearings =	119.00	ft
Number of Beams =	5	ft.
Beam Spacing =	10.00	ft
Slab Thickness = t_{sa} =	8.00	in
Structural Slab thickness= t_{ss} =	7.00	in
Slab- Concrete Strength = f'c =	4000	psi
Unit Weight of Concrete = w_c =	150.00	pcf
Modulus of Elasticity of Concrete = Ec = $w_c^{1.5}$*33*$f'c^{0.5}$/1000	3,834	ksi
Deck Reinforcing Yield Strength = f_y =	60,000	psi
Diaphragm or other loads placed on Non-composite Beam =	0.000	lbs/ft
Future Wearing Surface = FWS =	25	psf
Weight per foot of both parapets =	1000.0	lbs/ft
Haunch Width =	36.00	in
Haunch thickness =	1.00	in
Distance from Beam End to CL Bearing =	9.00	in

BEAM INFO

Beam Section Description =	Ohio 72"	
Depth of Beam = h =	72.00	in
Beam X-section Area = A =	956.00	in^2
Unit Weight of Concrete = w_c =	150.00	pcf
Section Modulus = S_b =	17,893	in^3
y_b =	34.43	in
Section Modulus = S_t =	16,396	in^3
y_t =	37.57	in
Concrete Strength at Release = f'ci =	5,000	psi
Modulus of Elasticity of Concrete = Ec = $w_c^{1.5}$*33*$f'c^{0.5}$/1000	4,287	ksi
Concrete Strength at 28 Days = f'c =	8,000	psi
Modulus of Elasticity of Concrete = Ec = $w_c^{1.5}$*33*$f'c^{0.5}$/1000	5,422	ksi
Top Flange Width = w_f =	36.00	in
Max thickness of Top flange (excluding fillets) =	6.00	in
fillet width (one side) =	3.00	in
web width =	8.00	in

SHEAR FORCES & BENDING MOMENTS

Beam Weight =	0.996	Kips/ft.
Slab Weight =	1.000	Kips/ft.
Haunch Weight =	0.038	Kips/ft.
Diaphragms =	0.000	Kips/ft.
Barriers =	0.200	Kips/ft.
FWS =	0.225	Kips/ft.
HS20 or HS25 =	HS-25	Livel Load
Moment Distribution Factor = DF_m =	1.818	wheels/beam
$DF_m/2$ =	0.909	Lanes/beam
Imact Factor =	0.205	Moment & Shear (conservative)

TRUCK

First Axle Weight =	10	Kips
Distance Between =	14	ft
Second Axle Weight =	40	Kips
Distance Between =	14	ft
Third AxleWeight =	40	Kips
Total Truck Load =	90	Kips
C.G. of Truck Load (from 2nd Axle) =	4.67	ft.
Lane Load Weight =	0.800	Kips/ft.
Roaming Point Load Shear =	32.5	Kips
Roaming Point Load Moment =	22.5	Kips

COMPOSITE PROPERTIES

Effective Web Width =	36.00	in	Art. 9.8.3.1
Effective Flange Width =	120.00	in	Art. 9.8.3.1 & 8.
Modular Ratio = n =	0.7071		
Transformed Flange Width =	84.85	in	
Transformed Flange Area =	593.97	in^2	
Transformed Haunch Width =	25.46	in	
Transformed Haunch Area =	25.46	in^2	

	Area = A	y_b	A*y_b	A*$(y_{bc}-y_b)^2$	I	I+A*$(y_{tc}-y_b)^2$
Beam	956.00	34.43	32915.08	259528.35	616055.99	875584
Haunch	25.46	72.50	1845.55	11869.59	2.12	11872
Slab	593.97	76.5	45438.68	389068.07	2425.38	391493
Summation	1,575.43		80199.31			1278950

	STRAND INFO	
Ultimate Stress = f's =	270,000	psi
Yield Strength = fy =0.9*f's =	243,000	psi
Initial Pretensioning = fs_i = 0.75*f's =	202,500	psi
Modulus of Elasticity = Es =	28,500	psi
Area of One Strand = As_1 =	0.167	in^2
Strand Diameter = d_s =	0.50	in
Transfer Length = 50*d_s =	25.00	in
Distance from Midspan to Draped Hold Down Point =	5.00	ft.
Initial Pretensioning Stress = 0.80*f_{pu} =	216000	psi
Number of strand draped =	13	
Average Draped Height =	60.00	in
(Total Hold Down Force)*(1.05 friction factor) =	44.56	Kips
Average Angle of Draped Incline =	5.17	degrees
	LOSSES	
Relative Humidity = RH =	75.0	%
Shrinkage Loss = SH =	5.75	ksi
Prestres after losses = Psi = (tot. strand area)*(0.69*f's) =	1617.8	kips
Avg. Conc. stress at C.G. of strands due to DL @ transfer = f_{cir}		
fcir = $P_{si}/A+P_{si}*e_o^2/I-(M_g+M_D)*e_o/I$ =	2.825	ksi
Elastic Shortening = ES = $E_s/E_{ci}*f_{cir}$ =	18.78	ksi
Conc. DL stress except initial DL = f_{cds} =		
$f_{cds} = M_s e_o/I+M_{SDL}*(y_{bc}-y_{bs})/I_c$ =	1.33	ksi
Concrete Creep Loss = $CR_c = 12*f_{cir}-7*f_{cds}$ =	24.60	ksi
Strand Relaxation =CRs = 5000-0.10ES-0.05*(SH+CRc) =	1.60	ksi
Total Prestress Loss at Transfer = ES =	18.78	ksi
Total Prestess loss at Transfer = $(ES)/f_{si}*100$ =	9.28	%
effective strand prestress force at transfer = $f_{sii} = f_{si}$-ES =	183.72	Ksi
Effective prestress force at transfer = P_{sii} = (Prestress Area)*f_{sii} =	1595.4	Kips
Total Prestess loss at Service = (SH+ES+CSc+CRs) =	50.74	ksi
Total Prestess loss at Service = (SH+ES+CSc+CRs)/f_{si}*100 =	25.06	%
effective strand prestress force at final = $f_{sfi} = f_{si}$-losses =	151.76	Kips
Effective prestress force at final = P_{sfi} = (Prestress Area)*f_{sfi} =	1317.9	Kips

Area of Composite Section = Ac =	1,575.43	in^2
Height of Composite Section =hc =	80.00	in
Dist. Composite Section Cg to btm of beam = y_{bc} =	50.91	in
Dist. Composite Section Cg to top of beam = y_{tg} =	21.09	in
Dist. Composite Section Cg to top of slab = y_{tc} =	29.09	in
Composite Moment of Inertia =I =	1278950	in^4
Composite Section Modulus to btm of beam = S_{bc} =	25124	in^3
Composite Section Modulus to top of beam = S_{tg} =	60632	in^3
Composite Section Modulus to top fiber of slab = S_{tc} =	62169	in^3

ASSUMED NUMBER OF STRNDS REQUIRED

bottom tensile Stresses at final = f_b =	3.991	ksi
Required stress @ Midspan bottom after losses = f_b-F_b =	3.454	ksi
ASSUMED strrand C.G. from bottom = y_{bs} =	4	in
(Typical C.G. = 5% for bulb Tees to 15% for AASHTO beams)		
ASSUMED Strand Eccentricity = y_b-y_{bs} =	30.43	in
ASSUMED bot. stress after losses = f_b =P_{se}/A + P_{se}*e_c/S_b => P_{se} =	1257.6	Kips
ASSUMED final losses % = (typ 25%) =	25.0	%
ASSUMED final losses =	50.63	ksi
ESTIMATED Number of Strands Required =	49.58	Strands

ALLOWABLE CONCRETE STRESSES

Compression @ transfer = 0.6*f'ci =	3.000	ksi
Tension @ transfer = 7.5*sqrt(f'ci) =	-0.530	ksi
Precast Compression @ final (Case I) = 0.6*f'c =	4.800	ksi
Slab Compression @ final (Case I) = 0.6*f'c =	2.400	ksi
Precast Compression @ final (Case II & III) = 0.4*f'c =	3.200	ksi
Slab Compression @ final (Case II& III) = 0.4*f'c =	1.600	ksi
Allowable Tensile Stresses = F_b = 6*sqrt*(f'c) =	-0.537	ksi
Modulus of Rupture = 7.5*SQRT(f'c) = f_r =	0.671	ksi

FLEXURAL STRENGTH PARAMETERS

strand factor = γ^* =	0.28	Art. 9.1.2
Conc. Strength Factor = 0.85-0.05*(f'c-4000)/1000<=0.65 = β_1 =	0.85	Art. 8.16.2.7
strength reduction factor = ϕ =	1.00	Art. 9.14
Maximum Reinforcement Limit (Ductility) = 0.36*β1 =	0.3060	Art 9.18.1

Section	Item	Bearing	Transfer	$H_d/2$	0.1*L	0.2*L	0.3*L	0.4*L	Depress	Random	0.5*L	Allowable Stresses: Tension	Allowable Stresses: Comp.	Allowable Stresses: Check / Remarks
	Distance =	0.00	1.33	3.33	11.90	23.80	35.70	47.60	54.50	50.00	59.50			
MOMENTS (kip-ft) & SHEAR (kips)	Beam Shear =	59.26	57.92	55.93	47.40	35.55	23.70	11.85	4.98	9.48	0.00			
	Beam Moment = M_g =	0.00	78.12	191.97	634.59	1128.16	1480.71	1692.24	1750.30	1717.81	1762.75			
	DL Moment @ release using full length = M_{gr} =	44.72	122.84	236.69	679.31	1172.88	1525.43	1736.96	1795.02	1762.53	1807.47			
	Slab+Haunch+Diaphragms Shear =	61.73	60.35	58.27	49.39	37.04	24.69	12.35	5.19	9.86	0.00			
	Slab+Haunch+Diaphragms Moment = M_s =	0.00	81.39	200.01	661.14	1175.36	1542.66	1763.04	1823.54	1789.69	1836.50			
	Barrier Weight Shear =	11.90	11.63	11.23	9.52	7.14	4.76	2.38	1.00	1.90	0.00			
	Barrier Weight Moment = M_b =	0.00	15.69	38.56	127.45	226.58	297.38	339.86	351.53	345.00	354.03			
	FWS Weight Shear =	13.39	13.09	12.64	10.71	8.03	5.36	2.68	1.13	2.14	0.00			
	FWS Weight Moment = = M_{ws} =	0.00	17.65	43.38	143.38	254.90	334.55	382.35	395.47	388.13	398.28			
	Live Load Turck Shear (w/o impact) =	82.94	81.93	80.42	73.94	64.94	55.94	46.94	41.72	45.13	37.94			
	Live Load Truck Moment (w/o impact) =	0.0	109.2	268.1	879.9	1545.6	1997.1	2262.4	2326.2	2292.8	2327.5			
	Live Load Turck Shear (w/o impact) =	80.10	76.66	76.58	67.81	56.46	46.07	36.64	31.60	34.65	26.15			
	Live Load Truck Moment (w/o impact) =	0.0	92.4	227.1	750.8	1334.7	1751.8	2002.1	2070.7	2032.3	2085.5			
	Maximum Shear Per Lane w/ Impact =	90.85	89.75	88.09	80.99	71.14	61.28	51.42	45.70	49.43	41.56			
	Maximum Moment Per Lane w/ Impact = M_{LL+I} =	0.0	119.7	293.6	963.8	1693.0	2187.6	2478.2	2548.1	2511.6	2549.5			
PROPERTIES	Draped Eccentricity (w/o debonding) =	14.011	14.361	14.886	17.135	20.258	23.381	26.504	28.315	27.134	28.315			
	Fraction of Group 1 debonding at this location =	1.00	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00			
	Fraction of Group 2 debonding at this location =	1.00	1.00	0.48	0.00	0.00	0.00	0.00	0.00	0.00	0.00			
	Eccentricity corrected for Debonding = e_c =	12.46	12.81	13.76	17.13	20.26	23.38	26.50	28.31	27.13	28.31			
	P_i Adjusted for Debonding = P_{iad} =	530.2	1472.7	1504.6	1595.4	1595.4	1595.4	1595.4	1595.4	1595.4	1595.4			
	P_e Adjusted for Debonding = P_{ead} =	437.9	1218.5	1242.9	1317.9	1317.9	1317.0	1317.9	1317.9	1317.9	1317.9			
STRESSES (ksi)	TRANSFER- Stresses in at BOTTOM = F_b =	0.894	2.512	2.572	2.741	2.688	2.730	2.867	2.990	2.906	2.981	-0.530	3.000	Stresses O.K
	TRANSFER- Stresses in TOP = F_o =	0.184	0.480	0.484	0.469	0.556	0.510	0.361	0.227	0.319	0.237	-0.530	3.000	Stresses O.K
	CASE I- Stresses in TOP = f_t =	0.830	0.950	1.065	1.508	2.036	2.342	2.425	2.371	2.414	2.390	-0.537	4.800	Stresses O.K
	CASE II- Stresses in TOP = f_t =	0.125	0.445	0.560	1.003	1.531	1.837	1.920	1.866	1.910	1.886	-0.537	3.200	Stresses O.K
	CASE III- Stresses in TOP = f_t =	0.587	0.727	0.785	1.006	1.270	1.423	1.465	1.438	1.459	1.447	-0.537	3.200	Stresses O.K.
	Stresses in BOTTOM = f_b =	0.763	1.963	1.814	1.182	0.257	-0.274	-0.515	-0.507	-0.525	-0.527	-0.537	4.800	Stresses O.K.
	CASE I- Stresses in top of SLAB = f_{ts} =	0.000	0.030	0.072	0.238	0.420	0.544	0.618	0.636	0.626	0.637		2.400	Stresses O.K
	CASE II- Stresses in top of SLAB = f_{ts} =	0.000	0.006	0.016	0.052	0.093	0.122	0.139	0.144	0.142	0.145		1.600	Stresses O.K
	CASE III- Stresses in top of SLAB = f_{ts} =	0.000	0.026	0.065	0.212	0.373	0.463	0.548	0.564	0.556	0.565		1.600	Stresses O.K
FLEXURE & REINF. CHECKS	Group 1 Load Factor = M_u =	0.0	510.5	1253.6	4129.0	7296.0	9501.1	10810.9	11149.0	10965.2	11192.0			
	Dist. From top slab to centroid of prestressing strand = d =	58.03	58.38	59.39	62.70	65.83	68.95	72.07	73.88	72.70	73.88			
	Rho = $A^*_s/(b^*d)$ = ρ^* =	0.001247	0.001240	0.001220	0.001154	0.001099	0.001050	0.001004	0.000979	0.000995	0.000979			
	Avg. Stress in prestressing steel at Ultimate = f^*_{su} =	262.5	262.6	262.7	263.1	263.4	263.7	264.0	264.1	264.0	264.1			
	depth of compression block = $A_s^* f^*_{su}/(0.85^* f'c^* b)$ = a =	5.54	5.54	5.55	5.60	5.61	5.61	5.62	5.62	5.62	5.62			
	Design Flexural Strength = $\phi A_s^* f^*_{su}{}^* d^*(1-0.6^* \rho^* f^*_{su}/f'c)$ = ϕMn =	10389.9	10457.5	10666.2	11393.8	12002.6	12611.6	13220.7	13573.9	13343.6	13573.9			Strength O.K.
	Ductility Reinforcement Index = $\rho^* f^*_{su}/f'c$ =	0.0818	0.0814	0.0801	0.0759	0.0724	0.0692	0.0663	0.0647	0.0657	0.0647			Ductility O.K.
	Comp from eff. Prestress & ext. loads = $P_{se}/A+P_{se}e_c/S_b$ = f_{pe} =	0.763	2.143	2.256	2.841	2.871	3.101	3.331	3.464	3.377	3.464			
	Non-comp DL moment = M_g+M_s = M_{dnc} =	0.00	159.50	391.98	1295.73	2303.52	3023.37	3455.28	3573.84	3507.50	3599.25			
	Cracking Moment = $(f_r+f_{pe})^*S_{bc}-M_{dnc}^*(S_{bc}/S_b-1)$ = M^*_{cr} =	3002.0	5827.5	5969.3	6409.2	6483.5	6674.2	6981.3	7212.6	7057.3	7202.3			
	$1.2^*M^*_{cr}$ =	3602.4	6993.0	7163.2	7691.0	7780.2	8009.0	8377.5	8655.1	8468.7	8642.8			Min. Reinf O.K

APPENDIX C

Post-Tensioned I-beam
Consplice PT, 1.0

Design Example

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FOR DEMONSTRATION ONLY                                          | SHEET   1 OF   13
                                                                | JOB NO. 1
PROGRAM: Consplice PT - V 1.0.17   by LEAP Software, Inc.       | BY bsc 09/1/2001
PHONE : TOLL-FREE 1-800-451-5327   TAMPA AREA: 813-985-9170     | CKD.
-------------------------------------------------------------------------------------
Project: Grad School

  Post-tensioned Tendon Information:
  P/T Tendon TendonType Fr.Coef    Wobble   PJ-Left PJ-Right Aset-L Aset-R Ul.Rlx
                                   (1/ft)    (kip)    (kip)   (in)   (in)  (ksi)
  ---------- ---------- ------- --------- -------- -------- ------ ------ ------
  PT1        31-0.5"(4.   0.250 0.0002000   835.11             0.375       (Calc)
  PT2        31-0.5"(4.   0.250 0.0002000   835.11             0.375       (Calc)


  Post-tensioned Tendon Path:
                                                              .......... POC ........
  P/T Tendon Path Type    Length  Rel.Dist Top Off    Var    #   Distance  Offset
                            (ft)     (ft)    (in)    Type          (ft)     (in)
  ---------- --------- --------- -------- -------- ------ ----- -------- -------
  PT1        Parabola    143.000    0.000    36.00
                                   71.500    68.00 Par-E
                                   71.500    36.00 Par-S
  PT2        Parabola    143.000    0.000    30.00
                                   71.500    62.00 Par-E
                                   71.500    30.00 Par-S
```

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Project: Grad School

Prestressing Losses

Stage 3: PS

POI	Location (ft)	Strand	Initial Force (kip)	Current Force (kip)	Percent Loss
Spl.000	0.00	Beam1 1	0.00	-2.64	0.00
		Beam1 3	0.00	-2.64	0.00
		Beam1 4	0.00	-2.64	0.00
		Beam1 6	0.00	-2.65	0.00
		Beam1 7	0.00	-2.65	0.00
		Beam1 8	0.00	-2.65	0.00
		Beam1 9	0.00	-8.00	0.00
Spl.010	14.30	Beam1 1	67.64	58.89	12.93
		Beam1 2	67.64	58.89	12.93
		Beam1 3	67.64	58.89	12.93
		Beam1 4	67.64	58.89	12.93
		Beam1 5	67.64	59.03	12.73
		Beam1 6	67.64	59.03	12.73
		Beam1 7	67.64	59.03	12.73
		Beam1 8	67.64	59.03	12.73
		Beam1 9	202.91	177.50	12.52
Spl.020	28.60	Beam1 1	67.64	59.73	11.69
		Beam1 2	67.64	59.73	11.69
		Beam1 3	67.64	59.73	11.69
		Beam1 4	67.64	59.73	11.69
		Beam1 5	67.64	59.81	11.57
		Beam1 6	67.64	59.81	11.57
		Beam1 7	67.64	59.81	11.57
		Beam1 8	67.64	59.81	11.57
		Beam1 9	202.91	179.69	11.44
Spl.030	42.90	Beam1 1	67.64	60.33	10.80
		Beam1 2	67.64	60.33	10.80
		Beam1 3	67.64	60.33	10.80
		Beam1 4	67.64	60.33	10.80
		Beam1 5	67.64	60.38	10.73
		Beam1 6	67.64	60.38	10.73
		Beam1 7	67.64	60.38	10.73
		Beam1 8	67.64	60.38	10.73
		Beam1 9	202.91	181.27	10.66
Spl.040	57.20	Beam1 1	67.64	60.69	10.27
		Beam1 2	67.64	60.69	10.27
		Beam1 3	67.64	60.69	10.27
		Beam1 4	67.64	60.69	10.27
		Beam1 5	67.64	60.71	10.23
		Beam1 6	67.64	60.71	10.23
		Beam1 7	67.64	60.71	10.23
		Beam1 8	67.64	60.71	10.23
		Beam1 9	202.91	182.21	10.20

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Spl.050	71.50	Beaml 1	67.64	60.80	10.11
		Beaml 2	67.64	60.80	10.11
		Beaml 3	67.64	60.80	10.11
		Beaml 4	67.64	60.80	10.11
		Beaml 5	67.64	60.81	10.09
		Beaml 6	67.64	60.81	10.09
		Beaml 7	67.64	60.81	10.09
		Beaml 8	67.64	60.81	10.09
		Beaml 9	202.91	182.48	10.06
Spl.060	85.80	Beaml 1	67.64	60.65	10.33
		Beaml 2	67.64	60.65	10.33
		Beaml 3	67.64	60.65	10.33
		Beaml 4	67.64	60.65	10.33
		Beaml 5	67.64	60.67	10.30
		Beaml 6	67.64	60.67	10.30
		Beaml 7	67.64	60.67	10.30
		Beaml 8	67.64	60.67	10.30
		Beaml 9	202.91	182.09	10.26
Spl.070	100.10	Beaml 1	67.64	60.25	10.92
		Beaml 2	67.64	60.25	10.92
		Beaml 3	67.64	60.25	10.92
		Beaml 4	67.64	60.25	10.92
		Beaml 5	67.64	60.30	10.85
		Beaml 6	67.64	60.30	10.85
		Beaml 7	67.64	60.30	10.85
		Beaml 8	67.64	60.30	10.85
		Beaml 9	202.91	181.04	10.78
Spl.080	114.40	Beaml 1	67.64	59.64	11.82
		Beaml 2	67.64	59.64	11.82
		Beaml 3	67.64	59.64	11.82
		Beaml 4	67.64	59.64	11.82
		Beaml 5	67.64	59.73	11.69
		Beaml 6	67.64	59.73	11.69
		Beaml 7	67.64	59.73	11.69
		Beaml 8	67.64	59.73	11.69
		Beaml 9	202.91	179.44	11.56
Spl.090	128.70	Beaml 1	67.64	58.81	13.05
		Beaml 2	67.64	57.72	14.66
		Beaml 3	67.64	58.81	13.05
		Beaml 4	67.64	57.72	14.66
		Beaml 5	67.64	58.95	12.84
		Beaml 6	67.64	58.95	12.84
		Beaml 7	67.64	58.95	12.84
		Beaml 8	67.64	59.98	11.32
		Beaml 9	202.91	180.39	11.10
Spl.100	143.00	Beaml 1	0.00	-2.68	0.00
		Beaml 3	0.00	-2.68	0.00
		Beaml 4	0.00	-2.68	0.00
		Beaml 6	0.00	-2.70	0.00
		Beaml 7	0.00	-2.70	0.00
		Beaml 8	0.00	-2.70	0.00

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Beam1 9	0.00	-8.12	0.00

FOR DEMONSTRATION ONLY | SHEET 5 OF 13
| JOB NO. 1
PROGRAM: Consplice PT - V 1.0.17 by LEAP Software, Inc. | BY bsc 09/1/2001
PHONE : TOLL-FREE 1-800-451-5327 TAMPA AREA: 813-985-9170 | CKD.

Project: Grad School

Post-Tensioning Losses

Stage 3: PT

POI	Location (ft)	Tendon	Initial Force (kip)	Current Force (kip)	Percent Loss
Sp1.000	0.00	PT1	755.09	736.22	2.50
Sp1.010	14.30	PT1	760.58	739.28	2.80
Sp1.020	28.60	PT1	766.03	737.48	3.73
Sp1.030	42.90	PT1	771.44	734.89	4.74
Sp1.040	57.20	PT1	776.82	734.27	5.48
Sp1.050	71.50	PT1	782.16	737.23	5.74
Sp1.060	85.80	PT1	787.47	744.34	5.48
Sp1.070	100.10	PT1	792.74	755.18	4.74
Sp1.080	114.40	PT1	792.23	762.70	3.73
Sp1.090	128.70	PT1	787.02	764.98	2.80
Sp1.100	143.00	PT1	781.85	762.31	2.50

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Project: Grad School

Stress Capacity Check

Stage 1: SLD-I_1

POI	Location (ft)	Max-Ten (ksi)	Loc	All-Ten (ksi)	Ratio	Max-Cmp (ksi)	Loc	All-Cmp (ksi)	Ratio
Sp1.000	0.00	0.00	Bm-B	0.54	999.99	0.00		N/A	999.99
Sp1.010	14.30	0.00		N/A	999.99	-1.35	Bm-B	-4.80	3.55
Sp1.020	28.60	0.00		N/A	999.99	-0.90	Bm-B	-4.80	5.33
Sp1.030	42.90	0.00		N/A	999.99	-0.97	Bm-T	-4.80	4.95
Sp1.040	57.20	0.00		N/A	999.99	-1.19	Bm-T	-4.80	4.03
Sp1.050	71.50	0.00		N/A	999.99	-1.26	Bm-T	-4.80	3.80
Sp1.060	85.80	0.00		N/A	999.99	-1.19	Bm-T	-4.80	4.03
Sp1.070	100.10	0.00		N/A	999.99	-0.97	Bm-T	-4.80	4.95
Sp1.080	114.40	0.00		N/A	999.99	-0.90	Bm-B	-4.80	5.33
Sp1.090	128.70	0.00		N/A	999.99	-1.35	Bm-B	-4.80	3.55
Sp1.100	143.00	0.00	Bm-B	0.54	999.99	0.00		N/A	999.99

Project: Grad School

Stress Capacity Check

Stage 1: SLD-I [PS + Perm]_1

POI	Location (ft)	Max-Ten (ksi)	Loc	All-Ten (ksi)	Ratio	Max-Cmp (ksi)	Loc	All-Cmp (ksi)	Ratio
Sp1.000	0.00	-------	----	-------	------	0.00		N/A	999.99
Sp1.010	14.30	-------	----	-------	------	-1.35	Bm-B	-3.60	2.66
Sp1.020	28.60	-------	----	-------	------	-0.90	Bm-B	-3.60	4.00
Sp1.030	42.90	-------	----	-------	------	-0.97	Bm-T	-3.60	3.71
Sp1.040	57.20	-------	----	-------	------	-1.19	Bm-T	-3.60	3.02
Sp1.050	71.50	-------	----	-------	------	-1.26	Bm-T	-3.60	2.85
Sp1.060	85.80	-------	----	-------	------	-1.19	Bm-T	-3.60	3.02
Sp1.070	100.10	-------	----	-------	------	-0.97	Bm-T	-3.60	3.71
Sp1.080	114.40	-------	----	-------	------	-0.90	Bm-B	-3.60	4.00
Sp1.090	128.70	-------	----	-------	------	-1.35	Bm-B	-3.60	2.66
Sp1.100	143.00	-------	----	-------	------	0.00		N/A	999.99

Project: Grad School

Stress Capacity Check

Stage 1: SLD-I [1/2(PS+Perm) + LL]_1

POI	Location (ft)	Max-Ten (ksi)	Loc	All-Ten (ksi)	Ratio	Max-Cmp (ksi)	Loc	All-Cmp (ksi)	Ratio
Sp1.000	0.00	-------	----	-------	------	0.00		N/A	999.99
Sp1.010	14.30	-------	----	-------	------	-0.68	Bm-B	-3.20	4.73
Sp1.020	28.60	-------	----	-------	------	-0.45	Bm-B	-3.20	7.11
Sp1.030	42.90	-------	----	-------	------	-0.48	Bm-T	-3.20	6.60
Sp1.040	57.20	-------	----	-------	------	-0.60	Bm-T	-3.20	5.38
Sp1.050	71.50	-------	----	-------	------	-0.63	Bm-T	-3.20	5.06
Sp1.060	85.80	-------	----	-------	------	-0.60	Bm-T	-3.20	5.38
Sp1.070	100.10	-------	----	-------	------	-0.48	Bm-T	-3.20	6.60
Sp1.080	114.40	-------	----	-------	------	-0.45	Bm-B	-3.20	7.11
Sp1.090	128.70	-------	----	-------	------	-0.68	Bm-B	-3.20	4.73
Sp1.100	143.00	-------	----	-------	------	0.00		N/A	999.99

Project: Grad School

Stress Capacity Check

Stage 1: SLD-I [PS + Perm + TU]_1

POI	Location (ft)	Max-Ten (ksi)	Loc	All-Ten (ksi)	Ratio	Max-Cmp (ksi)	Loc	All-Cmp (ksi)	Ratio
Sp1.000	0.00	0.00	Bm-B	0.54	999.99	0.00		N/A	999.99
Sp1.010	14.30	0.00		N/A	999.99	-1.35	Bm-B	-5.71	4.23
Sp1.020	28.60	0.00		N/A	999.99	-0.90	Bm-B	-5.71	6.35
Sp1.030	42.90	0.00		N/A	999.99	-0.97	Bm-T	-5.71	5.89
Sp1.040	57.20	0.00		N/A	999.99	-1.19	Bm-T	-5.71	4.80
Sp1.050	71.50	0.00		N/A	999.99	-1.26	Bm-T	-5.71	4.52
Sp1.060	85.80	0.00		N/A	999.99	-1.19	Bm-T	-5.71	4.80
Sp1.070	100.10	0.00		N/A	999.99	-0.97	Bm-T	-5.71	5.89
Sp1.080	114.40	0.00		N/A	999.99	-0.90	Bm-B	-5.71	6.35
Sp1.090	128.70	0.00		N/A	999.99	-1.35	Bm-B	-5.71	4.23
Sp1.100	143.00	0.00	Bm-B	0.54	999.99	0.00		N/A	999.99

Project: Grad School

Stress Capacity Check

Stage 1: SLD-I [PS + Perm + LL + TU + 0.5TG]_1

POI	Location (ft)	Max-Ten (ksi)	Loc	All-Ten (ksi)	Ratio	Max-Cmp (ksi)	Loc	All-Cmp (ksi)	Ratio
Sp1.000	0.00	0.00	Bm-B	0.54	999.99	0.00		N/A	999.99
Sp1.010	14.30	0.00		N/A	999.99	-1.35	Bm-B	-4.80	3.55
Sp1.020	28.60	0.00		N/A	999.99	-0.90	Bm-B	-4.80	5.33
Sp1.030	42.90	0.00		N/A	999.99	-0.97	Bm-T	-4.80	4.95
Sp1.040	57.20	0.00		N/A	999.99	-1.19	Bm-T	-4.80	4.03
Sp1.050	71.50	0.00		N/A	999.99	-1.26	Bm-T	-4.80	3.80
Sp1.060	85.80	0.00		N/A	999.99	-1.19	Bm-T	-4.80	4.03
Sp1.070	100.10	0.00		N/A	999.99	-0.97	Bm-T	-4.80	4.95
Sp1.080	114.40	0.00		N/A	999.99	-0.90	Bm-B	-4.80	5.33
Sp1.090	128.70	0.00		N/A	999.99	-1.35	Bm-B	-4.80	3.55
Sp1.100	143.00	0.00	Bm-B	0.54	999.99	0.00		N/A	999.99

Project: Grad School

Moment Capacity Check

Stage 1: LFD-I_1

POI	Location (ft)	Mu (kip-ft)	Phi*Mn (kip-ft)	1.2*M-Cr (kip-ft)	Cap/Mu
Sp1.000	0.00	0.00	0.00	-1097.28	999.99
Sp1.010	14.30	1191.17	5282.32	1203.37	4.39
Sp1.020	28.60	2117.60	5282.32	1203.37	2.49
Sp1.030	42.90	2779.31	5282.32	1203.37	1.90
Sp1.040	57.20	3176.29	5282.32	1203.37	1.66
Sp1.050	71.50	3308.53	5282.32	1203.37	1.60
Sp1.060	85.80	3176.05	5282.32	1203.37	1.66
Sp1.070	100.10	2778.84	5282.32	1203.37	1.90
Sp1.080	114.40	2116.90	5282.32	1203.37	2.50
Sp1.090	128.70	1190.23	5282.32	1203.37	4.39
Sp1.100	143.00	-1.17	0.00	-1097.28	0.00 *

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Project: Grad School

Shear Capacity Check

Stage 1: LFD-I_1

POI	Location (ft)	Vu (kip)	Conc/Mom (kip-ft)	Av/S (in)	Vc (kip)	Vs (kip)	Cap/Vu
Sp1.000	0.00	-92.55	0.00	0.0000	125.49	0.00	1.22
Sp1.010	14.30	-74.04	1191.17	0.0000	251.49	0.00	3.06
Sp1.020	28.60	-55.53	2117.60	0.0000	121.78	0.00	1.97
Sp1.030	42.90	-37.02	2779.31	0.0000	74.73	0.00	1.82
Sp1.040	57.20	-18.50	3176.29	0.0000	72.15	0.00	3.51
Sp1.050	71.50	0.01	3308.53	0.0000	72.15	0.00	999.99
Sp1.060	85.80	18.52	3176.05	0.0000	72.15	0.00	3.51
Sp1.070	100.10	37.03	2778.84	0.0000	74.75	0.00	1.82
Sp1.080	114.40	55.55	2116.90	0.0000	121.81	0.00	1.97
Sp1.090	128.70	74.06	1190.23	0.0000	251.66	0.00	3.06
Sp1.100	143.00	92.57	-1.17	0.0000	125.50	0.00	1.22

Project: Grad School

Design Summary

Stage 1

POI	Location (ft)	.... Min C/D Ratios .... Moment	Shear	St-Ten	St-Cmp
Sp1.000	0.00	999.99	1.22	999.99	999.99
Sp1.010	14.30	4.39	3.06	999.99	2.66
Sp1.020	28.60	2.49	1.97	999.99	4.00
Sp1.030	42.90	1.90	1.82	999.99	3.71
Sp1.040	57.20	1.66	3.51	999.99	3.02
Sp1.050	71.50	1.60	999.99	999.99	2.85
Sp1.060	85.80	1.66	3.51	999.99	3.02
Sp1.070	100.10	1.90	1.82	999.99	3.71
Sp1.080	114.40	2.50	1.97	999.99	4.00
Sp1.090	128.70	4.39	3.06	999.99	2.66
Sp1.100	143.00	0.00*	1.22	999.99	999.99